Transportation
Library
TA
1
.A526

TRANSACTIONS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS.

(INSTITUTED 1852.)

TRANSPORTATION LIBRARY
UNIVERSITY OF MICHIGAN

VOL. LIX.

DECEMBER, 1907.

Edited by the Secretary, under the direction of the Committee on Publications.

Reprints from this publication, which is copyrighted, may be made on condition that the full title of Paper, name of Author and page reference are given.

NEW YORK:

PUBLISHED BY THE SOCIETY.

1907.

Entered according to Act of Congress, by the AMERICAN SOCIETY OF CIVIL ENGINEERS, in the Office of the Librarian of Congress, at Washington.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

CONTENTS.

PAPERS.

No.		PAGE
1051	**THE LOWER COLORADO RIVER AND THE SALTON BASIN.**	
	By **C. E. Grunsky**	1
	Discussion:	
	By W. G. Price	52
	Rutger B. Green	53
	T. G. Dabney	54
	W. de H. Washington	55
	J. A. Ockerson	58
	C. E. Grunsky	60
1052	**LIBERATION OF AIR IN SIPHONS.**	
	By **Charles Anthony, Jr.**	63
	Discussion:	
	By P. M. Pritchard	65
	Charles Anthony, Jr.	65
1053	**THE LAWS OF PROPORTIONING CONCRETE.**	
	By **William B. Fuller** and **Sanford E. Thompson**	67
	Discussion:	
	By James L. Davis	144
	H. de B. Parsons	149
	Allen Hazen	149
	Samuel Tobias Wagner	150
	R. Feret	152
	W. F. Dennis	157
	Richard H. Gaines	159
	William B. Fuller and Sanford E. Thompson	169
1054	**A SOLUTION OF THE PROBLEM OF DETERMINING THE ECONOMIC SIZE OF PIPE FOR HIGH-PRESSURE WATER-POWER INSTALLATION.**	
	By **Arthur L. Adams**	173
	Discussion:	
	By W. L. Butcher	178
	W. E. Buck	179
	P. E. Stevens	182
	C. L. DeMott	183
	E. Kuichling	184
	William J. Roberts	187
	Charles D. Marx and C. B. Wing	188
	James H. Wise	189
	E. W. Myers	191
	Asahel C. Toll	192
	F. G. Baum	193
	Arthur L. Adams	194

No.		PAGE.
1055	**THE REINFORCED CONCRETE BRIDGE ACROSS THE HUDSON RIVER AT SANDY HILL, NEW YORK.**	
	By **William H. Burr**	195
	Discussion :	
	By Henry H. Quimby	203
	C. L. Slocum	204
	D. W. Krellwitz	206
	E. W. Stern	206
	William H. Burr	206
1056	**THE EFFECTS OF THE SAN FRANCISCO EARTHQUAKE OF APRIL 18TH, 1906, ON ENGINEERING CONSTRUCTIONS. REPORTS OF A GENERAL COMMITTEE AND OF SIX SPECIAL COMMITTEES OF THE SAN FRANCISCO ASSOCIATION OF MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS.**	
	REPORT OF GENERAL COMMITTEE	208
	REPORT OF COMMITTEE ON THE GEOLOGY OF THE EARTHQUAKE	216
	REPORT OF COMMITTEE ON FIRE AND EARTHQUAKE DAMAGE TO BUILDINGS	223
	REPORT OF COMMITTEE ON THE EFFECT OF THE EARTHQUAKE ON WATER-WORKS STRUCTURES	245
	REPORT OF COMMITTEE ON THE EFFECT OF THE EARTHQUAKE ON LOCAL TRANSPORTATION AND LIGHTING PLANTS	256
	REPORT OF COMMITTEE ON THE EFFECT OF THE EARTHQUAKE ON RAILWAY STRUCTURES	258
	REPORT OF COMMITTEE ON THE EFFECT OF THE EARTHQUAKE ON HIGHWAY STRUCTURES	263
	Discussion :	
	By Edwin Duryea, Jr	264
	Franklin Riffle	266
	H. H. Wadsworth	270
	Emile G. Perrot	270
	Luther Wagoner	272
	Eugene W. Stern	273
	J. K. Freitag	277
	W. W. Harts	282
	E. A. Rix	283
	W. F. Whitaker	284
	A. M. Bienenfeld	284
	Arthur L. Adams	285
	S. G. Hindes	285
	Loren E. Hunt	285
	Bernard Bienenfeld	285
	J. P. Carlin	286
	Charles E. Goad	289
	R. B. Green	292
	C. E. Moore	293
	A. L. A. Himmelwright	296
	Langdon Pearse	310
	C. Derleth, Jr	311
	J. D. Galloway	323
1057	**ELECTRIC RAILWAYS: AN INFORMAL DISCUSSION.**	
	By George Gibbs	330
	W. W. Follett	333
	J. T. Noble Anderson	334

No.		PAGE.
1058	**PAVEMENTS: AN INFORMAL DISCUSSION.**	
	By GEORGE W. TILLSON	336
	J. H. HAYLOW	341
	CLIFFORD RICHARDSON	342
	HORACE ANDREWS	346
	GARDNER S. WILLIAMS	348
	LANSING H. BEACH	349
	E. M. T. RYDER	351
	WILLIAM H. LAWTON	353
	PHILIP W. HENRY	356
	B. BIENENFELD	361
	NELSON P. LEWIS	362
	EDWARD P. NORTH	364
1059	**WATER SUPPLY: AN INFORMAL DISCUSSION.**	
	By GEORGE W. FULLER	367
	F. HERBERT SNOW	376
	D. D. CLARKE	384
	GEORGE A. SOPER	387
	WILLIAM J. BALDWIN	388
	GARDNER S. WILLIAMS	388
	J. T. NOBLE ANDERSON	389
	WILLIAM B. FULLER	390
	J. WALDO SMITH	391
	PABLO SOLÍS and OCTAVIO GUZMÁN	392
	S. BENT RUSSELL	394
	M. R. SHERRERD	396
	F. W. BLACKFORD	398
	GEORGE A. JOHNSON	398
	GEORGE W. FULLER	400
1060	**GAS ENGINES: AN INFORMAL DISCUSSION.**	
	By JAMES CHRISTIE	402
	ROBERT HEYWOOD FERNALD	405
	TIMOLEON GEELEN	413
	J. M. BUCHANAN	428
1061	**COMPARISON BETWEEN RAINFALL AND RUN-OFF IN THE NORTH-EASTERN UNITED STATES.**	
	By **John C. Hoyt**	431
	Discussion:	
	By E. C. MURPHY	484
	CHARLES A. HOLDEN	488
	H. C. FRANKENFIELD and ALFRED J. HENRY	489
	THADDEUS MERRIMAN	491
	RAPHAEL ZON	493
	C. C. VERMEULE	496
	H. K. BARROWS	499
	J. C. STEVENS	502
	M. O. LEIGHTON	504
	F. H. NEWELL	508
	A. P. DAVIS	509
	N. C. GROVER	510
	F. W. HANNA	511
	JOHN C. HOYT	514

MEMOIRS OF DECEASED MEMBERS.

	PAGE.
SYLVANUS THAYER ABERT, M. AM. SOC. C. E.	521
CHARLES BLACKWELL, M. AM. SOC. C. E.	531
LOUIS GUSTAVE FREDERIC BOUSCAREN, M. AM. SOC. C. E.	533
JOHN EUGENE CHENEY, M. AM. SOC. C. E.	537
JOSEPHUS CONN GUILD, M. AM. SOC. C. E.	540
DUNKIN WIRGMAN HEMMING, M. AM. SOC. C. E.	543
CHARLES EZRA HEQUEMBOURG, M. AM. SOC. C. E.	545
JACOB HAYS LINVILLE, M. AM. SOC. C. E.	549
GEORGE BENSON NICHOLSON, M. AM. SOC. C. E.	556
MERRITT HARRISON ROGERS, M. AM. SOC. C. E.	562
ALBERT JOHNSTONE CAMPBELL, ASSOC. M. AM. SOC. C. E.	563
JOSEPH WILLIAM ZIPPERLEIN, ASSOC. AM. SOC. C. E.	565

PLATES.

PLATE.		PAPER.	PAGE.
I.	Views of Dry Bed of Colorado River, and of Salton Sea	1051	5
II.	Views of Alamo River, and the Ruined Field near New River	1051	17
III.	Enlarged Lower Mexican Heading, Imperial Canal	1051	27
IV.	Wooden Gate and By-Pass at Lower Mexican Heading, Imperial Canal	1051	33
V.	Views of the Barranca of New River	1051	35
VI.	Views of the Barranca of New River from Calexico	1051	37
VII.	The Waste-Gate at Sharp's Heading, Mexico	1051	39
VIII.	Head-Gate of Canal at Sharp's Heading; and Brush Mat at Lower Mexican Heading	1051	41
IX.	Dam and By-Pass at Lower Mexican Heading	1051	43
X.	Apparatus Used in Volumetric Tests of Concrete; and Implements Used in Making Beams	1053	77
XI.	Machine for Breaking Beams; and Apparatus for Testing the Permeability of Concrete	1053	111
XII.	List of Beam Tests (Table 8)	1053	113
XIII.	List of Beam Tests (Table 8)	1053	113
XIV.	Apparatus for Volumetric Tests of Density of Concrete; and Variations in Volume of Cement Paste with Different Percentages of Water	1053	139
XV.	Diagram Showing the Economic Size of Pipe for High-Pressure Water-Power Installations	1054	181
XVI.	General Drawing of Bridge over the Hudson River at Sandy Hill, New York	1055	197
XVII.	Centering for Sandy Hill Bridge	1055	199
XVIII.	Longitudinal Sections, Sandy Hill Bridge	1055	201
XIX.	Transverse Sections, Sandy Hill Bridge	1055	203
XX.	Steel Ribs for Sandy Hill Bridge	1055	203
XXI.	Reinforced Concrete Highway Bridge over Tracks of New York, New Haven and Hartford Railroad	1055	205
XXII.	View of Sandy Hill Bridge, Completed	1055	207
XXIII.	Relief Map of California; and Offset in Fence Line in Portola Valley	1056	217
XXIV.	Furrow along Fault Line of Earthquake; and Displacement of Soil	1056	219
XXV.	Earthquake Damage to Colonnades and Library of Stanford University	1056	225

PLATE.		PAPER.	PAGE.
XXVI.	Earthquake Damage to Library and Museum of Stanford University	1056	227
XXVII.	Earthquake Damage to Memorial Arch, etc., Stanford University	1056	229
XXVIII.	Earthquake Damage to City Hall, San Francisco, and to Terra Cotta in James Flood Building	1056	231
XXIX.	Fire Damage to Granite Columns and Marble Stairway	1056	233
XXX.	Fire Damage to Marble, Terra Cotta, etc	1056	235
XXXI.	Fire Damage to Terra Cotta, Brick Piers, and Elevator Appliances	1056	237
XXXII.	Fire Damage to Columns in Buildings	1056	239
XXXIII.	Earthquake Damage to Concrete Block Construction; and Fire Damage to Inner Court of Mills Building	1056	241
XXXIV.	United States Mint, and Hall of Justice, San Francisco	1056	243
XXXV.	Views Showing Fire Damage to Buildings of Type 2 Construction	1056	245
XXXVI.	Earthquake Damage to Frame Buildings, San Francisco; and to Agnews' Asylum, near San José	1056	245
XXXVII.	Earthquake Damage to Court House, Santa Rosa, and Church in Oakland	1056	245
XXXVIII.	Earthquake Damage to San Francisco Post Office	1056	245
XXXIX.	Earthquake Damage to Tower of Ferry Building, San Francisco; and Fire Damage to Lath and Plaster Partition	1056	245
XL.	Map of San Francisco Bay Region Showing Fault Lines, and Location of Important Water-Works Structures	1056	247
XLI.	Fissure in Slope of Dam; and Cracked and Displaced Rip-Rap on Fault Line at Dam of San José Water Company, near Saratoga	1056	249
XLII.	Transverse Crack, and Hole, in Dam of San José Water Company at Saratoga	1056	251
XLIII.	Water Tower, Santa Clara Water-Works, in Process of Erection	1056	253
XLIV.	Water Tower, Santa Clara Water-Works, Before and After Its Destruction	1056	255
XLV.	San Andreas and Crystal Springs Pipe Lines	1056	255
XLVI.	Typical Breaks in Pilarcitos Pipe Line	1056	255
XLVII.	Surface Indications of Fault Line along Pilarcitos Pipe Line	1056	255
XLVIII.	Map of San Francisco, Showing Areas Within Which Subsidence Occurred During the Earthquake	1056	255
XLIX.	Views Showing Displacement of Ground by Earthquake	1056	255
L.	Views Showing Damage to 30-In. Pipe Line	1056	255
LI.	Closure of Cable Slots in Burned District, San Francisco	1056	257
LII.	Damage to Piers of Pajaro River Bridge by Earthquake	1056	259
LIII.	Damage to Railroad Bridge; and Overturned Passenger Train	1056	261
LIV.	Highway Bridge near Point Reyes, Marin County, Cal	1056	271
LV.	Views of Salinas Highway Bridge	1056	313
LVI.	Spreckels' Sugar Mill	1056	319
LVII.	Views of Wreck of Alder Creek Bridge	1056	325
LVIII.	Views of Gualala Bridge, Injured by Earthquake	1056	327
LIX.	Map of the United States Showing the Principal Drainage Areas, and the Rainfall	1061	435
LX.	Measurements of Flow, Area, and Velocity of Susquehanna River, at Harrisburg, Pa., 1897 to 1906	1061	519

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 1051.

THE LOWER COLORADO RIVER AND THE SALTON BASIN.*

By C. E. Grunsky, M. Am. Soc. C. E.

With Discussion by Messrs. W. G. Price, Rutger B. Green, T. G. Dabney, W. de H. Washington, J. A. Ockerson, and C. E. Grunsky.

Early in 1905 it became apparent to the engineers of the United States Reclamation Service that the Colorado River would change its low-water course; that it would thereafter send its low-water flow and a part of its flood volume inland into the Salton Basin instead of oceanward into the Gulf of California. Ever since that time the Reclamation Service has been an interested observer of the progress made by the canal people (whose operations are responsible for the change of the river's course) in their efforts to curb the river. The observations of the Reclamation Service engineers have been extended also to the changes which have been taking place along the new watercourse.

This paper is based upon the results of these examinations, and is in a measure a transcript of the writer's official reports on this subject.

The point in Mexico where the canal company made the diversion which threatens to become the permanent river channel is

* Presented at the meeting of March 20th, 1907.

about 4 miles south from the boundary line between California and Mexico. It was visited by the writer twice in 1905 and three times in 1906.

In the spring and summer of 1905, the Colorado River—in Lower California at this point, which is some 12 miles below Yuma—enlarged a small artificial channel or cut which connected the river with the Imperial Canal, and also enlarged the canal to such an extent that this so-called canal heading and the canal became the river's low-water channel. The river, in November, 1905, during a flood stage, made this heading and portions of Alamo and New Rivers and certain sections of the canal its main channel. The river thereupon flowed inland for a year, discharging into the Salton Sea instead of holding to its original southerly course with discharge into the Gulf of California, where it belongs, and where its waters have gone for many centuries. The river was turned from this inland course on November 4th, 1906, but could be held in its proper channel only until December 7th, when the protection works gave way. The river is again flowing into the Salton Sea. The Salton Sea is the water of the Colorado River which has accumulated in California in the bottom of the depression (about 278 ft. below sea level at the lowest point) known to geographers as the Colorado Desert, but recently referred to as the Salton Basin.

The canal, which for the present has become a river, was constructed primarily for the irrigation of lands in California in the Imperial Valley, as the southern portion of the Salton Basin is called. This canal is frequently called the Imperial Canal. It is the property of the California Development Company and of a subsidiary Mexican corporation through which the California Development Company has been conducting operations in Mexico.

The original water diversion from the Colorado River, by the California Development Company for the irrigation of lands in California, was made in the southeastern corner of that State within a few hundred feet of the international boundary; but the operations which led to a change of the river's course were entirely within the territory of Mexico, and are presumed to have been conducted under concessions granted by that country to the canal companies.

A continuation of the inland flow of the Colorado River would in time bring disaster to the Imperial Valley. It would further damage the property of the Southern Pacific (railroad) Company, which has already sustained great financial loss, and would ultimately ruin all improved lands in the Salton Basin as well as the lands along the lower sections of the Colorado River. It would modify or destroy the irrigation possibilities along this river below its cañon.

Salton Basin.

"Salton Basin" is the name applied to the region in southeastern California and northern Lower California which will ultimately be submerged if the Colorado River continues to discharge inland instead of into the Gulf of California.

This region extends northward to a point a few miles beyond Indio, and toward the south overlaps the international boundary some 15 to 20 miles. The length of the basin which is thus threatened with submersion is about 100 miles from north to south; its surface area is about 1 950 sq. miles, and its mean width nearly 20 miles.

The lowest point of this basin, at the former salt beds, now more than 70 ft. under water, is 278 ft. below sea level.

The rainfall in the country adjoining this great depression is so slight—only a few inches per annum—that practically no run-off reaches its lowest points. All the water now there, covering more than 400 sq. miles, is inflow from the Colorado River.

The Southern Pacific Company, about 1876, built its main line across this depressed area, dropping to a lowest point near Salton with an elevation—or rather greatest depression—about 265 ft. below sea level. Its main line, from a point several miles north from Indio to near Flowing Well, some 3 miles east from Imperial Junction (formerly Old Beach), is below sea level.

At the salt beds, the New Liverpool Salt Company established salt works, apparently a remunerative enterprise, but these beds were completely submerged early in 1905.

At Indio, and southward from this point to near the salt beds, wherever water has been obtainable from adjoining streams or by boring wells, as near Coachella, Artesia and Mecca, the land is being improved, and homes have been established. Before the river

was turned, these were just beginning to fall a prey to the rising waters of Salton Sea, and are now again menaced.

Since the California Development Company's irrigation work from the Colorado was undertaken, the valley south of the basin's greatest depression has been known as the Imperial Valley. Here towns quickly sprang up, and Imperial, Brawley, Holtville, Calexico and more recently El Centro have become centers of activity.

Mr. W. F. Holt, President of the Imperial Valley Water Users' Association, states that in January, 1905, the population of Imperial Valley was 10 000. It is probable that the number of persons who had homes, real property, or other interests at stake in the valley in its most prosperous days exceeded this number.

Calexico is at sea level, Imperial about 60 to 70 ft. below sea level, and Brawley about 110 ft. below sea level. The surface of Salton Sea is now about 207 ft. below sea level. The rise of the water, therefore, would have to be about 90 ft. before the country near Brawley would be inundated.

Although the Imperial Valley is not yet covered by the waters of Salton Sea, much damage has been done to the lands in this valley by the river water in its flow from south to north along the Alamo and New Rivers. These, particularly New River, were not intended by Nature to carry such volumes of flow as the river discharged into them after it broke from its proper course at the Lower Mexican Heading. Therefore, there were times in 1905 and 1906 when great areas along these streams were flooded, and other areas would have been inundated, except for the protection they received by strenuous work on long lines of dikes or levees. Unfavorably located tracts were sacrificed to save the more fortunate districts, and every effort was bent to increase the carrying capacity of New River along which flood damage was greatest.

The canal system was not planned for such unforeseen flood conditions. Structures were inadequate to safeguard properly the supply to the canals. Constant vigilance was required to keep out the undesirable flood, and to keep, in the canals, an adequate supply for the needs of the irrigators. In fact, many critical periods have been passed. Any untoward accident at Sharp's Heading during the flood stage of the river would have rendered the entire Imperial country untenable. An accident there, such as a wash-out of

FIG. 1.—DRY BED OF COLORADO RIVER, BELOW LOWER MEXICAN HEADING OF IMPERIAL CANAL. AUGUST 26TH, 1906.

FIG. 2.—SUNSET ACROSS SALTON SEA. FROM NEW SALTON RAILROAD STATION, AUGUST 29TH, 1906.

the waste-gate which drops water some 20 ft., would have deprived the canal system of water—a serious situation for a region in which even the water for domestic use is supplied by the canals.

But the danger of a general inundation passed when New River, in June, 1906, had carried the head of a great barranca, which it was cutting in the plain, past Calexico and Mexicali, and had drained much of the overflowed region lying to the south of the boundary line. Then the question arose, would the river be turned before this back cutting reached the Alamo above Sharp's Heading, drawing the water from the canals; or would the dreaded water famine have to be faced even while the whole river went rushing by? This question is again pending, now that the engineers at the Lower Mexican Heading have failed to hold the river in its proper channel.

The topographic features of the country make it plain why all this has happened. The river, like all turbid streams, is still building up the alluvial country through which it flows. It drops some of its silt wherever it sends its water. At flood stages, when there is overbank flow, the rate of silt deposit is most rapid near the river. The frequent inundation of the bank lands adds to their elevation more than is added to land farther away. Therefore, here, as should be expected, the land slopes away from the river.

The Colorado River flows southerly in a direction in which the general fall of the ground surface is only about 1½ ft. per mile, which the river in its meanderings cuts down to an effective fall of about 1 ft. per mile.

Toward Volcano Lake, southwest from the river, the general surface gradient is 2 ft. or more per mile; and westward, in the direction paralleling Alamo River, it is nearly 3 ft. per mile to a point near Calexico. Thence northward into Salton Basin, on lines of greatest slope, the country falls away at the rate of from 4 to 5 ft. per mile.

These gradients, which are cross-country gradients and do not show the fall of the water surface following the meanderings of the several watercourses, make it plain that the river while flowing inland must cut deep into any yielding material. Its proper fall was about 120 ft. in its course from Yuma to the Gulf, 80 miles distant, whereas it now falls some 325 ft. to the Salton Sea which,

in the general direction of the water's flow, is only from 5 to 10 miles farther removed from Yuma. The excess of energy represented in these figures has been largely expended by the river in deepening and enlarging its new bed.

Channel cutting and deepening of the river-bed, if the river be not turned, will in time extend up stream far above Yuma, and, in such event, will continue until the rising waters in Salton Sea reduce the excessive river gradients that prevail during the Sea's low stages. Ultimately, if the river flows inland unchecked, the basin waters will overflow at or near Volcano Lake. The river's discharge will then be into the southeastern extremity of the Sea, about 30 miles westward from Pilot Knob, at an elevation somewhere in the neighborhood of 30 ft. above sea level, and the process of delta building at the river's outfall point will recommence on the lake margin and will continue until the river at Pilot Knob is restored to its original height. The time will then come when conditions of slope will be reversed, when there will be greater fall toward the Gulf than toward Salton Sea, and then finally the river will itself resume its course to the sea. Centuries would probably elapse before this would occur.

Early Explorers of the Lower Colorado River.

An expedition sent out by Cortes, Viceroy of New Spain, in 1539, under Francisco de Ulloa sailed northwestward from Acapulco to the head of the Gulf of California. This expedition established the fact that Lower California was not an island, but a peninsula, and that there was probably a great river discharging into the head of the Gulf; but the river was not entered.

A year later, Francis Vasco de Coronado, then Governor of New Galicia, the province north of New Spain, was commissioned by Mendoza, the successor of Cortes as Viceroy, to explore the country to the northwest with the specific purpose of reaching the famed Seven Cities of Cibola. The exploration was to be both by land and sea. Coronado himself led the land party, and Captain Hernando de Alarçon was placed in charge of three ships in which he sailed up the Gulf of California and found the great river, as had been expected. Coronado had already started northward overland. It was evidently expected that the two branches of the expedition

would be in communication with each other, from which fact it appears to be certain that the existence of the great river was more than mere conjecture.

Alarçon succeeded in overcoming the difficulties of entering the mouth of the river, and won the friendship of the Indians, who assisted him in ascending the river in boats, which were for the most part towed with ropes by the Indians. His first journey up stream occupied 15 days, and he returned down stream in less than 3 days. Preparations were then made for a longer absence from his ships, and the river was again ascended probably to some point nearly 100 miles above Yuma. A cross was erected, and messages were left for Coronado, should he reach the river.

Meanwhile, Coronado, who was far to the northeast, had detached Captain Melchior Diaz from the main body of explorers to attempt a junction with Alarçon. Diaz reached the Colorado and learned that boats had ascended the river to a point 3 days' journey below. He reached that spot and found letters from Alarçon who, at that time, however, had already departed from his main camp at the mouth of the river. Diaz made a short exploration to the west of the Colorado River, but the country was too forbidding to promise any result of value.

Concerning the subsequent fate of Diaz, Mr. F. S. Dellenbaugh, in his entertaining narrative, "The Romance of the Colorado River," says:

"A dog belonging to the camp pursued the little flock of sheep that had been driven along to supply the men with meat, and Diaz on his horse dashed toward it, at the same time hurling a spear. The spear stuck in the ground instead of striking the dog, and the butt penetrated the Captain's abdomen, inflicting under the conditions a mortal wound. The men could do nothing for him except carry him along, which for 20 days they did, fighting hostile natives all the time. Then he died. On the 18th of January (1541) they arrived without their leader at the settlement from which they had started some three months before."

Sixty-three years later (in 1604) Don Juan de Onate, a wealthy governor of New Mexico, reached the Colorado River, at the mouth of the Bill Williams Fork, and traveled down the stream to the Gulf.

Padre Eusibio Francisco Kino was the next traveler whose visit

to the mouth of the Colorado is recorded. He crossed the river in 1701, about a day's journey above its mouth, and in the spring of 1702 was at its mouth.

Ugarte reached the mouth of the Colorado by way of the Gulf of California in 1721, and it was again visited in 1744 by Fernando Consay, who ascended the Gulf looking for suitable mission sites.

In 1771 Padre Garces descended the Gila to the Colorado, and continued down the latter probably to its mouth. In 1776 he again journeyed down the river from Yuma to the Gulf.

In 1826 Lieut. R. W. H. Hardy, of the British Navy, in the schooner *Bruja,* sailed up the Gulf and met with some mishaps to his vessel at the mouth of the river. He mistook the channel which now bears the name Hardy's Colorado for the main stream, and thought the river proper was the Gila. He named the islands which lie in the head of the Gulf, Montague and Gore.

Two trappers, James P. Pattie and his father, who had in 1825 made a long journey, paralleling the cañon of the Colorado from the mouth of the Gila up to the river's sources in Colorado and Wyoming, again reached the river in 1827, and, being robbed of their horses by the Indians, decided to make a trip down the Colorado to the settlements supposed to be at its mouth. They made the trip leisurely, but could find no settlements, and were finally turned back by the fierce currents of the ebb and flood tides in the mouth of the river, and after many hardships reached the Pacific Coast.

Ever since 1850 there has been an American settlement on the river at Yuma, and the river has been explored repeatedly above and below that point. Lieut. G. H. Derby, in the winter of 1850-51, made an exploration of the waters at the mouth of the river, and his expedition was met by boats sent down the river by Major Heintzelman then stationed at Yuma.

The first steamboat on the river below Yuma commenced making trips about 1853.

In 1857 Lieut. J. C. Ives, in a small steamboat, *The Explorer,* brought to the river in sections, ascended to Black Cañon, and continued his explorations overland to points on the Grand Cañon.

The records of some of the early explorations are meager, but, when taken in their entirety, the important fact seems to be well

established that the Colorado River has not, for any protracted period in historic times, abandoned its course into the Gulf of California, and that at no time since its discovery, nearly 400 years ago, has there been any general submersion of the Colorado Desert.

This knowledge is valuable because quite recently—in geologic time—there has been a great fresh-water lake covering the desert to an elevation about 30 ft. above sea level. This lake has left its borders plainly marked upon the ground. The beach line is clearly defined and can be traced around the ancient lake bed. An interesting fact in reference to this basin was discovered in 1905, when an outline topographic survey was made of the basin, once full of water, by the U. S. Geological Survey (W. Carvel Hall being in charge of the field work). It was found that subsequent to the time when water had filled this basin there had been an upheaval of the country along its eastern margin to the extent of 20 ft. or more. The beach line, which at the western margin of the ancient lake is about 30 ft. above the sea, is in the neighborhood of 50 ft. above on the eastern margin. No information is at hand as to whether this upheaval was gradual or sudden; but, as there do not appear to be any secondary beach lines below the well-defined ancient beach, it appears reasonably certain that the upheaval was subsequent to the existence of the lake.

Earthquakes.

In connection with this probable uplift on the Colorado River side of the Salton Basin, it may be noted that it is believed that a southern continuation of the great fault line along which there was some movement on April 18th, 1906, both north and south of San Francisco, causing an earthquake on that day, passes southward into the Gulf of California through the Salton Basin. At about 4 P. M. on the day of the San Francisco earthquake, there was an earthquake in the Imperial country, apparently most severe near Brawley. It is reported to have caused considerable breaking down of the high vertical alluvial banks along the newly eroded barranca of New River.

On November 9th, 1852, there was an earthquake in the southeastern portion of California, in southwestern Arizona and in parts of Lower California, of which Dr. Blake (1853) says:

"The earthquake of 1852 (November) was sufficiently violent to break off a piece of Chimney Peak and to open fissures in the clay of the desert."

Referring to the same earthquake, Lieut. J. C. Ives (Exploration of 1857) relates that in November, 1852, a schooner at anchor 30 miles above the mouth of the Colorado River, floating in 14 ft. of water, had an earthquake experience. There was a rumbling sound and a shock, and then suddenly sand arose, leaving the vessel aground with only 3 ft. of water around it. It was floated off by the succeeding spring tides.

The Ancient Lake of the Colorado Desert.

That there was at one time a lake or inland sea of large extent in the Colorado Desert, covering somewhat more than the area which is below sea-level, was recognized by Dr. William P. Blake, the geologist of the exploring expedition for a railroad route from the Mississippi westward to the Pacific Ocean (1853 to 1855). The character of the water which filled the basin is known from the shells still found there in large numbers. It was fresh water, or water only slightly brackish.

There is no doubt that this ancient lake was cut off from the north end of a long narrow arm of the ocean which extended into California to near Indio, and was a northerly continuation of the Gulf of California. At the period, in geologic history, when the Gulf of California thus extended inland some 150 miles farther than at present, the Colorado River had its mouth upon the eastern shore of this great bay at or near the point where Yuma is now located. The river's water surface at that point was then practically at sea level; it is now, varying with the water stage (high or low), from 117 to 130 ft. above sea level.

The river then, as now, carried seaward a large quantity of silt. This quantity is now in the neighborhood of 50 sq. mile-ft. per year (1 ft. deep over 50 sq. miles); in the past, during ages of greater rainfall and greater erosion, it may have been much greater than this. Whatever the rate at which this material was delivered, it has been sufficient in the course of time to build up the present river delta, to push a delta cone entirely across the long bay to the

base of the Cocopah Mountains in Lower California, to push the head of the Gulf southward to a point about 80 miles from Yuma (more than 100 miles by river), and to build up the valley lands in Yuma to their present elevation, about 100 ft. above the sea. The river has thus built a barrier of silt across what was once a great submarine valley. This barrier, which was not the work of a day, but was many centuries in building, trapped the water of the upper end of the prehistoric gulf and created the inland lake.

Subsequent to the formation of this lake, the building up of the river's delta cone continued until the lowest point of the ground (not channel beds) on the broad, flat barrier which now separates the Gulf of California from the Salton Basin was raised to about 30 ft. above sea level.

This elevation of 30 ft. was then, approximately at least, the elevation of the lake water surface.

Referring to this ancient lake, Dr. Blake (1855) says:

"The former existence of a fresh water lake in the northern part of the desert is shown, not only by the extensive deposits of alluvial or lacustrine clay, containing fossils, but by the existence of extended shore lines and beaches along the sides of the bordering mountains."

After alluding to the Indian tradition of great floods which had brought devastation to this region, he says:

"The present outflows, though but very slight, are probably similar, and yet it is possible that the interior of the desert might be deluged at the present day, provided no elevation of the land has taken place, and the river should remain at a great height for a long time—long enough to cause the excavation of a deep channel for New River."

The Indian tradition relating to the ancient Salton Sea is thus given by Dr. Blake:

"The great water (*agua grande*) covered the whole valley and was filled with fine fish. There were also plenty of ducks and geese. * * * Their fathers lived in the mountains and used to come down to the lake to fish and hunt. The water gradually subsided, *'poco,' 'poco'* (little by little), and their villages were moved down from the mountains into the valley it had left. They also said that the waters once returned very suddenly and overwhelmed many of their people and drove the rest back to the mountains."

The River Delta.

It may well be imagined that, in the upbuilding of the delta lands of the Colorado, the river made many changes in its point of outflow. Its mouth swung back and forth between Lake and Gulf, and there may have been many long periods in which its entire flow was either to the one or to the other.

The Colorado Desert was dry or nearly dry once before the long dry period of our own day. This is shown by the mark which the inrushing water that refilled the basin has scored across the valley lands. Then, as to-day, the water cut deep into the alluvial soil, taking a course down the easterly side of the valley and discharging into the remnant of a sea which probably had its southerly margin some miles to the southward of Brawley. The remnant of one of the bluffs of this channel, now a well-defined bank of gradual slope, can be seen some miles east from Imperial.

When an examination is made of the light alluvial material with which the river has built its delta cone, and the ease is considered with which the river cuts the banks in this material, one may well be surprised that it has not long ago resumed its flow into the Colorado Desert, to which course it is invited by the steeper gradients of the surface of the ground in that direction, as above noted. The main force resisting this change is probably the obstruction offered to channel cutting in bank lands by the dense and luxuriant plant growth which is found wherever moisture is available. The growth of vegetation is rank; grasses, weeds, willows, cottonwood and other trees cover the stream banks with a dense thicket through which water cannot flow with sufficient current to cut a channel. Should it happen that an open spot is found and the flow is concentrated across such a spot, then drift will there begin to run, which, by lodging in the brush beyond, aids the silt dropped by the water and soon builds up the open spot to even greater height than the remainder of the bank. Unusual conditions and exceptional combinations of circumstances are required to permit a sufficient concentration of the flowing water, and for such a time and distance that channel cutting can be caused by the overbank flow, anywhere near the main stream.

Whenever a delta slough, a recipient of overflow water, shows a tendency to cut its head closer to the river channel, the resulting

increased overbank discharge toward this point causes rapid accretion to the bank land and a piling up of silt and drift that quickly shuts off the supply of water necessary to advance the new channel farther toward the parent stream.

The various flood-fed streams which flow down the delta cone of the Colorado River westward from its proper channel are therefore without direct connection with this channel. Their waters accumulate in the brush thickets on the bank lands from which they emerge in small streams some miles from the river. A number of such small streams unite to form large watercourses such as the Pescadero River and the Rio Paredones.

The Alamo River was of the same character as these other streams before it was connected with the Colorado artificially.

Reference to the map, Fig. 1, will give a better idea of the location of these several watercourses than can be given in words. It may be stated, however, that the several channels which cross the international boundary line from Mexico into the State of California, were by early travelers referred to as New River or branches of New River. The name Alamo is now applied to the watercourse which, heading in Mexico some 8 to 12 miles southeast from Pilot Knob, was fed by overbank flow from the Colorado River, holds a western and northwestern course along the base of the sand hills lying to the north until it enters California about 40 miles in an air line westward from the Colorado; continuing northward, it empties into the Salton Sea. This river, or some portion of it, is sometimes referred to as Carter's River and as Salton River.

In the mesquite and arrow-weed thicket at the original head of the Alamo, there was an occasional accumulation of so much water, and submersion of so much land, that the locality was called "the Lagoons" (*Las Lagunas*). Although these lagoons received water at practically every high-water stage of the river, they did not always yield enough to the Alamo River to produce a flow throughout the river's entire length. In other words, there were many years in which the Alamo did not discharge any water into the lowest portion of the Salton Basin.

The lagoons, in addition to feeding the Alamo, appear also to have been one of the sources of supply for the Rio Paredones. This stream, which is also known as Quail River and as Middle River

(*Rio Medio*) takes a southwesterly course down the backbone of the delta cone. The water which overtops the right bank of the Colorado at points from 5 to 10 miles below Pilot Knob is the principal source of supply for the feeders of Rio Paredones. The Spanish word, *Paredon,* means big wall. The name of the river, therefore, implies high banks or a well-defined channel, which is true at least of certain portions of the watercourse. It flows practically on the crest of the divide between the Gulf and the lake, and terminates at Volcano Lake, which is probably the remnant of a broad outflow channel which at one time connected the ancient Salton Sea with the Gulf. At freshet stages of the Colorado the Paredones channel may be surcharged, and it then spills water overbank to the south into feeders of Pescadero River and to the north into feeders of New River.

Volcano Lake is crowded against a low, narrow bench at the base of the Cocopah range of mountains. Its bed is at an elevation of about 15 to 20 ft. above sea level and its high-water stage is about 22 to 25 ft. above sea level. The bench land to the west has probably been built up by the mud volcanoes which are still active along the western margin of the lake. These are of the well-known type of solfataras, and present the usual characteristics of hot mud springs. Well-traveled Indian trails to several pools of moderately hot mud indicate their popularity as a cure for rheumatic ills. They are located at the base as well as on the top of the raised area above designated as a bench. Some of them are submerged at high lake stages. Their activity seems to be somewhat intermittent. During the one night that the writer camped among them, their puffing, blowing and sputtering were vigorous and quite general in the evening and during the early hours of the night, but they were very quiet early in the morning. The outlying springs frequently take the form of miniature volcanoes, perfectly cone-shaped, about 3 to 5 ft. high, with steam issuing from small (6 to 9-in.) circular openings in the top. In many cases a cone of this character has a shattered side, and the bubbling mud is held in place by a rim of caked material extending out from the side of the original cone. In other cases the boiling mud forms large pools in which there are spots of special activity. The bursting of gas bubbles, sometimes

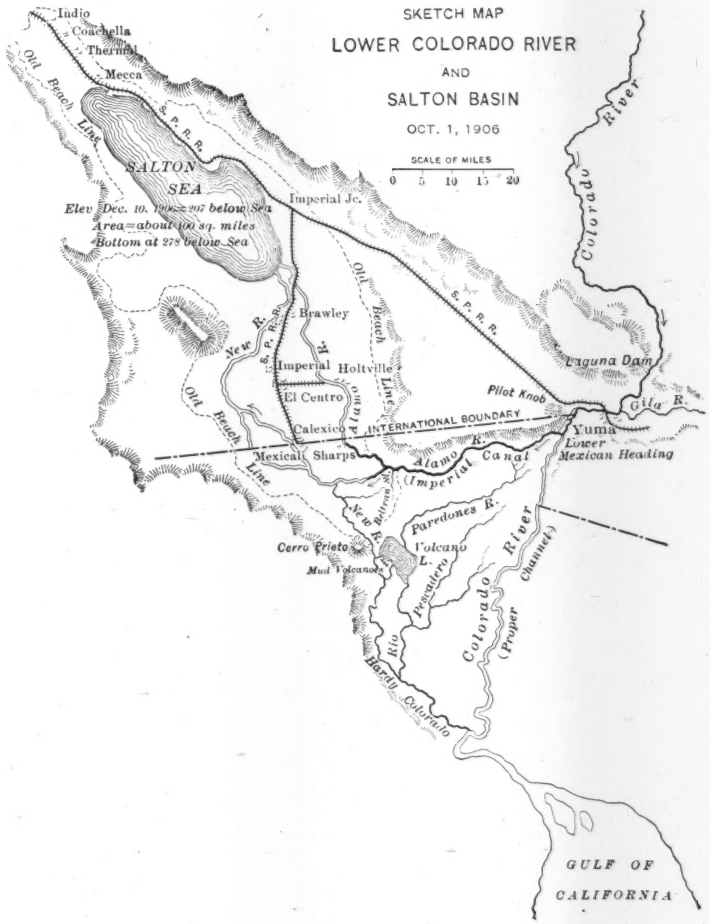

SKETCH MAP
LOWER COLORADO RIVER
AND
SALTON BASIN
OCT. 1, 1906
SCALE OF MILES
0 5 10 15 20
Indio
Coachella
Thermal
Mecca
Old Beach Line
S. P. R. R.
SALTON SEA
Elev Dec. 10, 1906=207 below Sea
Area=about 100 sq. miles
Bottom at 278 below Sea
Imperial Jc.
Colorado River
Brawley
New R.
Imperial
Holtville
El Centro
Calexico
INTERNATIONAL BOUNDARY
Mexicali
Sharps
Laguna Dam
Pilot Knob
Gila R.
Yuma
Lower Mexican Heading
Alamo Canal
Imperial
New River
Paredones R.
Cerro Prieto
Mud Volcanoes
Volcano L.
Pescadero
Colorado River (Proper Channel)
Rio Hardy Colorado
GULF OF CALIFORNIA

FIG. 1.

a foot or more in diameter, throws the mud several feet into the air in great thin semi-liquid slabs.

The main outlet of Volcano Lake is Hardy's Colorado, which flows from the southwest extremity of the lake and joins Colorado River just above its entrance into the Gulf. Pescadero River, already referred to, is tributary to Hardy's Colorado. Hardy River, where it leaves the lake, is a channel several hundred feet wide and probably 15 or 20 ft. deep at high stages.

Volcano Lake has a secondary outlet toward the north. This is the channel to which the name New River has in recent times been restricted. New River, at the lake margin and for some distance toward the north, is not a waterway of large capacity. Its bed is not as low as that of Hardy's Colorado. Less than ¼ mile from the lake, this channel sends off a branch, Salt Slough, toward the south, to a union with Hardy's Colorado. The mud volcanoes lie upon the high ground between the Salt Slough and Volcano Lake.

The part of New River near Volcano Lake, Salt Slough, Volcano Lake, and Hardy's Colorado, are possibly remnants of the waterways which connected the ancient Salton Sea with the Gulf. New River northward from this point was originally a rather ill-defined channel, here and there lost in mesquite thickets. It held a course northward from Volcano Lake to the salt beds of the Salton Basin. Its flow was irregular, depending upon the duration of the high-water stages of the Colorado River and the amount of the overflow which reached the Alamo and Rio Paredones. The river is the recipient of the water draining from the low area which lies between these two rivers. Therefore, it receives the overbank discharges of the Alamo toward the south, and of the Paredones toward the north. These discharges were ordinarily of no great moment, but since the river has made the Alamo its main channel and the natural capacity of the Alamo has been restricted by the structures maintained at Sharp's Heading, there has been a large flow of water overbank from this river. The lower section of New River has become the channel for the delivery of this water into Salton Sea. New River crosses the international boundary about 8 miles west from the point where this boundary line is crossed by the Alamo. The small towns, Mexicali and Calexico, are adjacent to

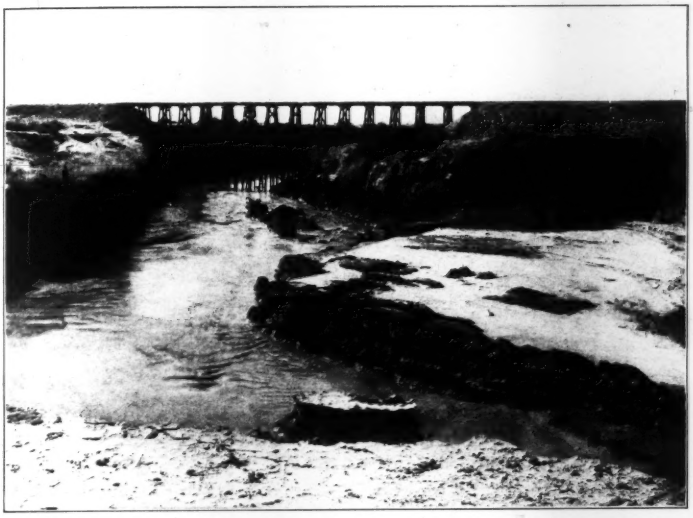


FIG. 1.—ALAMO RIVER, AT HOLTVILLE RAILROAD BRIDGE, AUGUST 31ST, 1906.

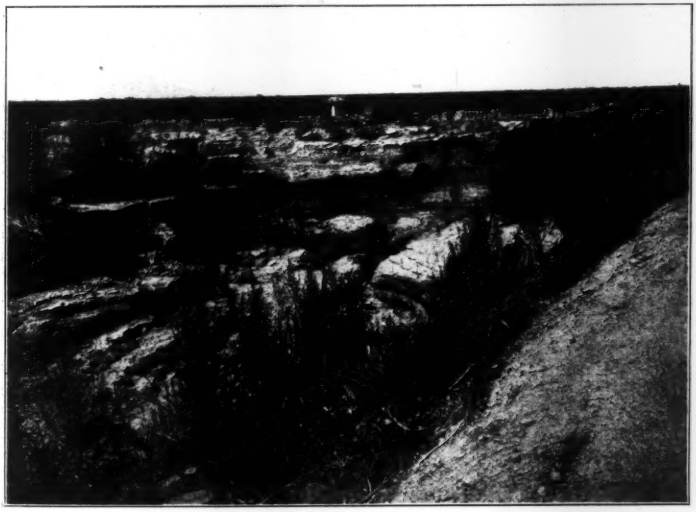


FIG. 2.—THE RUINED FIELD, NEAR NEW RIVER, ABOUT 4 MILES NORTHWEST OF CALEXICO, SEPTEMBER 2D, 1906.

each other upon the two sides of the boundary line and on the east bank of New River. Before the changes in the character of the river-bed which took place during 1906, the river had no well-defined channel at this point; it occupied a broad swale, pushing its way through the open spots in the scattered growth of mesquite, and the water which it ordinarily received was for the most part lost by seepage and evaporation before reaching the salt beds.

But there have been occasional seasons in which extraordinary flood conditions prevailed; and Alamo and New Rivers became flowing streams and the salt beds a lake of water. Lieut. Bergland learned from residents on or near the Colorado (1875-76) that in the years 1840, 1852, 1859, 1862, and 1867 there had been great floods of this character in the New River country, and refers to an account by a traveler in the desert in 1862 who reports having seen in the Salton Basin a great lake 60 miles long and 20 miles wide. This, of course, is an over-estimate of size, as it would have taken the river many years to send overbank enough water to flood so large a portion of the basin. To the high-water years mentioned by Lieut. Bergland others may now be added. Professor Blake refers to the year 1849 as having been reputed the first in which the Colorado sent an overbank flow into the New River country. He recognized at once, however, that this must have happened many times before. In February, 1891, the Colorado rose to an unusually high stage, the water at that time being contributed mainly by the Gila and its tributaries. It overtopped its banks below Yuma, and submerged large areas along the Alamo and New Rivers. Enough water reached the salt beds to submerge them and to cause much speculation by the uninformed as to the source of supply for the lake which had suddenly made its appearance. The years 1903, 1905, and 1906 are also to be added to the flood years, but the original conditions of temporary overbank flow during freshet stages have been modified, the river having elected during these years to discharge inland instead of to the Gulf.

The Irrigation of the Colorado Desert.

The reclamation of the Colorado Desert in California by irrigation is not a new idea. Prompted by the project of Dr. O. M. Wozencraft and his associates, the Legislature of California in 1859

adopted a memorial to Congress requesting that 3 000 000 acres of land, the southeastern portion of California, be ceded to the State for reclamation by irrigation. It was proposed, on the basis of this expected cession, to authorize the construction of irrigation works by a private corporation, and the location of a canal partly in Mexico was apparently not considered a formidable obstacle. On this application of the State, the Committee in the House of Representatives made a favorable report (1862), but the measure failed to pass, presumably due to the objection of the Commissioner of the Land Office, who looked upon the scheme as one put forth for the prime advantage of its promoters.

It is of interest, however, in connection with this application for a cession of land, to learn from the testimony of Dr. Wozencraft that the lands of the Colorado Desert were to be reclaimed by irrigation, and that his engineer, Mr. Ebenezer Hadley, County Surveyor of San Diego County, in reporting on this project, recommended a canal location which was practically identical with that actually adopted more than 40 years later. Mr. Hadley said among other things that irrigation was practicable, as water from the Colorado River flows there now. He stated that he had found by actual survey that there was a fall of 5 ft. per mile available along the proposed canal route. He called attention to the necessity of carrying water through Mexican territory. He proposed a canal, 25 ft. wide and 10 ft. deep, which was to tap the river at the point of rocks adjoining Pilot Knob and immediately above the international boundary line. This is probably the exact point at which the California Development Company has recently constructed a concrete headgate. Referring to the names of the natural channels of the New River country, Mr. Hadley says:

"Water having passed through one of these channels from Colorado River soon after our acquisition of the country it was called New River; the other may be justly called Old River."

On this subject of New River, Dr. Blake, on December 5th, 1853, wrote:

"We passed two canal-like channels or wide gullies in the surface, with mesquite trees growing in the bottom, and evidences of the presence of water at a former period. Shells of 'anodonta' and small spiral shells were abundant. These channels probably com-

municate with the two lagoons, and may be the bed of the stream called New River, so-called from the fact of its sudden appearance in 1849. At that time the Colorado River was very high, and broke over a part of its banks between the mouth of the Gila and the head of the Gulf. The water flowed inland, running backward through the desert toward the center of the valley once occupied by the ancient lake; thus proving the valley to be lower than the banks of the river at the place of the overflow. Major W. H. Emory, and party, who crossed the desert from the Colorado to Carizo Creek in 1846, does not mention this stream or any indication of its pre-existence. He mentions, however, that he was informed by a Mexican that a running stream would be found a league west of the Alamo (*Alamo Mocho,* a camping place).

"Parties were sent out to search for the water, but no stream could be found. The appearance of the stream in 1849 was a subject of general surprise and wonder; and was an unexpected relief to the many emigrant parties who crossed the desert that year. It is the general belief that this overflow was the first of its kind, but it has evidently often taken place before."

Dr. Blake, further, says of New River:

"It derives its supply from the Colorado River only at high water, and is probably fed through numerous and ramified channels of slight depth. The precise locality of the entrance of the stream is not known; this renders it more probable that there is not a well-defined channel at the Colorado, but that the water spreads over the banks and afterward collects in one channel."

The feasibility of diverting water from the Colorado at or below the lower end of the Grand Cañon was investigated in 1875 and 1876 by Lieut. Eric Bergland, Corps of Engineers, U. S. Army, who acted under the direction of Lieut. George M. Wheeler. His investigations disclosed the difficulty of locating a diverting canal on United States territory, and he reached the conclusion that the most feasible route for a canal would be below the boundary line. The following is from his report:

"Below the Purple Hills there are no formidable mountain ranges on the west side of the river, except a short detached range, called the Cargo Muchacho, and Pilot Knob. The divide between these mountains is about 278 ft., while the altitude of the water-surface at Fort Yuma is 120 ft. A canal through this opening would therefore require a cutting of nearly 160 ft., and, besides, would have to cut through the sand ridge west of Pilot Knob in

order to gain lower ground more rapidly than by keeping to the north of the sand-hills. Even in this case the length of the canal from Fort Yuma to the point where the surface of the ground is of the same altitude as the water-surface at the latter place would be at least 30 miles long.

"These conclusions are arrived at from data obtained from the Texas Pacific Railroad surveys, and also from levels run by my party from Hanlon's Ferry over the divide north of Pilot Knob. In order that the canal should be entirely within the California boundary, it must cross the divide to the north of Pilot Knob. This, as has already been shown, would necessitate a long, deep cutting, partly through rock, and a passage through the sand ridge. This passage could only be effected by means of a flume or tunnel, to protect it from filling up with sand. A canal from some point below the boundary would be more practicable and less expensive in construction and maintenance. A line of levels was run from Fort Yuma to Indian Wells, and to the dry canal beyond. * * * * * One of the branches of New River leaves the Colorado near Algodon station, and this artificial (natural?) channel might be utilized in the construction of a canal for diverting the water of the river into the depressed area to the northwest. The exact course of this canal cannot be determined without further surveys, but it seems probable that it would be necessary to run it below the boundary-line nearly as far as Seven Wells in order to avoid the sand ridge. From this point it could bend more or less toward the north, according to the downward slope of the surface in that direction. The amount of cutting required is difficult to estimate without further surveys, but it would doubtless be moderate, as the water flows into this area from the river when it overflows its banks."

Notwithstanding this report, which was discouraging to all who had hoped that water could be supplied to the desert in a canal located, throughout its entire length, in United States territory, the promoters of irrigation enterprise continued to look with longing eyes toward a project which would make the desert a garden spot. The region to be irrigated was well known to travelers. It was an uninhabited, grassless, barren plain. Its productiveness when water was made available could be taken for granted. It is an almost rainless region, with warm summer and mild winter climate. Its flat surface and light uniform gradients rendered it particularly attractive to the irrigator.

Finally, Mr. C. R. Rockwood and his associates conceived and

carried out the idea of excavating canals on privately owned lands in Mexico. This was done apparently without any franchise from the Mexican authorities. The water diverted from the Colorado River in California, in a short canal only a few hundred feet long, arriving at the boundary line near the Colorado River, was allowed to fill an extension of this canal on Mexican territory which discharged into the Alamo River a few miles farther on. The natural flow of the Alamo was westward and northward into California where the water was required for irrigation. The canals to distribute this water to the several districts to be covered could be taken out of the Alamo most conveniently in Mexico, and so a secondary diversion was made from this stream, and the heads of several canals which flow into the Imperial country were located south of the line in Mexico. All necessary works in Mexico were located upon land owned or controlled by the canal companies, and the entire course of the Alamo in Mexico was through this land. International questions were thus suppressed or held in the background.

Apparently with this idea of avoiding the necessity of securing concessions from Mexico, and of suppressing international questions, the Colorado River Irrigation Company was organized (1892), and surveys were made and works were planned to deliver water in Mexico and in the Salton Basin for irrigation and domestic use; but the necessary financial backing was lacking, and this company, after several years of preliminary work, failed. Through the further efforts of Mr. Rockwood, a second corporation, the California Development Company, was organized (April 24th, 1896) under the laws of New Jersey, and became the successor in interest of the properties of the earlier company. At the head of this company, managing it and directing its fiscal affairs, was the late Mr. A. H. Heber. He remained at its head, except for two years, 1900-1902, until the reorganization in June, 1905, which put the control into the hands of the Southern Pacific Company.

To carry out the plan of constructing canals in Mexico on land controlled by the Development Company, a Mexican corporation was formed—*"La Sociedad de Irrigacion y Terrenos de la Baja California (Sociedad Anonym)"*—which is presumably the same corporation which subsequently received a concession from Mexico

—"*La Sociedad de Riegos y Terrenos de la Baja California* (*Sociedad Anonym*)." All the corporation stock of this Mexican corporation was, and presumably still is, owned or controlled by the California Development Company.

The Mexican Company thereupon acquired from Guillermo Andrade, who was owner of a colonization concession, a tract of land, 100 000 acres in area, which extended along or near the international boundary westward from the Colorado River to beyond the point where the Alamo crosses the line, excluding along the boundary only the barren sand hills projecting over the line from California. There was also an agreement made for the purchase by the Company from Andrade of an additional 100 000 acres.

On the land thus brought under the control of the Mexican company, canal construction was carried on in Mexico, without interference by Mexican authorities. Water was made available for the Imperial lands in 1902 (a small quantity only as early as 1901).

Although Mexico did not make any objections to the use of land within that country as a right of way for carrying water into the United States, there is, it is understood, a protest on file in the Department of State against the diversion of water from the Colorado River (November, 1901); and in this protest the possibility of the whole flow of the river being turned by the proposed irrigation works is pointed out.

The original notice of intention to divert water from the Colorado River in California bears the date, April 25th, 1899. The quantity of water claimed under this notice was 10 000 cu. ft. per sec., and the place of the proposed diversion was referred to as about 1¼ miles above the international boundary line.

The canal of the California Development Company, as originally constructed, had its head in California at Hanlon's, or Hanlon's Crossing, about 100 yd. north from the international boundary. The canal was cut out from the river at an oblique angle, and its flow was controlled by a wooden structure placed across the canal several hundred feet from the river bank. On a falling river the head of the canal and the gate were obstructed by silt deposits, and it became difficult to keep the water flowing from the river into the canal.

These silt accumulations in the head of the canal interfered with its successful operation, and water shortage resulted in the Imperial Valley in the fall of 1903, and continued throughout 1904.

The company in 1904 sought to secure from Congress a grant of a specific quantity of water from the Colorado, a navigable stream. In this it failed, and it also failed to have Congress declare the river non-navigable. Upon application to the Mexican government, however, a concession was made by that country to the affiliated Mexican corporation, under which operations on Mexican territory were legalized, and the right to divert water in Mexico for use upon lands in the United States was granted.

The original canal was carried from California into Mexico, and paralleled the river for a distance of about 4 miles, being generally ¼ to ½ mile from the river bank. Its course then changed from south to southwest and west. It passed close along the south base of the sand hills which hang on the south spur of Pilot Knob, and its first section terminated with a length of about 11 miles at the lagoon. Thence the canal water was carried in an artificial channel northwestward to a well-defined section of Alamo River, and flowed in this river to near the point about 40 miles west of the Colorado, where the Alamo enters California. There, about 1 mile south from the international boundary, water is diverted from the Alamo into a number of irrigation canals.

This point, known as Sharp's Heading, had been selected as the proper place for the main diversion of the water from the natural channel into the canals which were to command the Imperial country. Other diversions were maintained, for a time, farther down stream—at the Eastside levee, several miles above Holtville, for the irrigation of District No. 5, and at the Rose levee, several miles below Holtville, for the irrigation of lands in the northeastern portion of District No. 1. But imperfect control of the inflow into the canal head at the Colorado, with consequent flood conditions along the Alamo when the Colorado was high caused much surplus water to pass Sharp's Heading, which resulted in the repeated destruction of these levees or diverting dams, and led to their abandonment. The entire section of the Alamo below Sharp's Heading was thereafter used as a waste channel.

It would be useless to attempt a minute description of the works constructed from time to time to keep the improved lands supplied with water. It will suffice to state that at present there is no other point than Sharp's Heading on the Alamo at which a diversion is made from this channel for the irrigation of the Imperial lands.

Districts Nos. 5 and 7, which lie upon the east side of the Alamo, have canals heading at Sharp's. Imperial Canal No. 1 is the original main canal, on the flow of which most of the land between the Alamo and New Rivers (Districts Nos. 1 and 4) is dependent, and it too has its head at Sharp's; so, also, does the Wisteria Canal which at present supplies water only to lands in Mexico belonging to the California and Mexico Land and Cattle Company. Districts Nos. 6 and 8, lying on the west side of New River, are at present entirely without water.

The Mexican concession above referred to was granted, under date of May 17th, 1904, and received the approval of the Mexican Congress in the following month. Under this concession the Mexican corporation was granted permission to divert water to the amount of 10 000 cu. ft. per sec. from the Colorado River on Mexican territory, subject, however, to the use of one-half of this water on demand upon lands in Mexico.

The right was also granted to receive this water from a canal heading in California north of the boundary line, and to convey it in canals or natural channels through Mexico. It is required under this concession that plans and profiles of canals already built in Mexico be filed, and that additional works be approved by the Mexican authorities before construction. Other provisions usual in such cases are that the work done is to be subject to the inspection of an engineer, and the company is required to pay to Mexico $300 per month perpetually. Bridges are to be constructed for road crossings. The company is permitted to import, free of duty, instruments, machinery and the like for use on canal work. The company is exempted from taxation for 10 years, except in the matter of stamps. It is prohibited from selling or mortgaging its property to any foreign government or State, or from taking such government or State into partnership. A deposit of $10 000 in Mexican bonds was required. It was also made a condition that

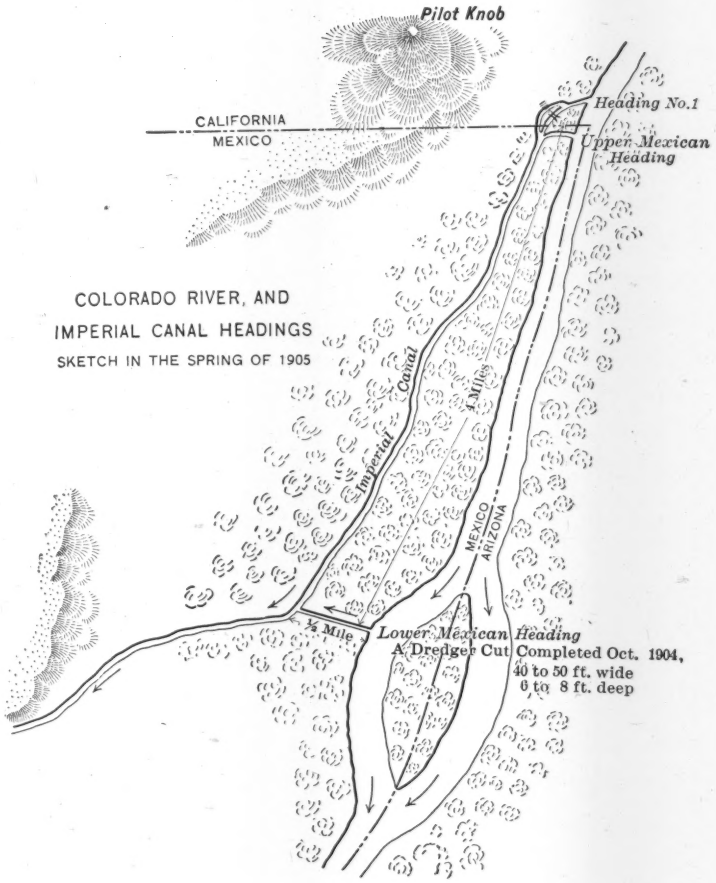

Pilot Knob
CALIFORNIA
MEXICO
Heading No.1
Upper Mexican Heading
COLORADO RIVER, AND
IMPERIAL CANAL HEADINGS
SKETCH IN THE SPRING OF 1905
Imperial Canal
4 Miles
MEXICO
ARIZONA
½ Mile
Lower Mexican Heading
A Dredger Cut Completed Oct. 1904,
40 to 50 ft. wide
6 to 8 ft. deep

FIG. 2.

the corporation holding this concession must be a Mexican corporation, but that any or all stockholders might be foreigners.

Operating under this concession, there was constructed just south of the international boundary a cut less than 100 yd. long from river to canal. This connected with the canal below its original timber headgate, but did not materially improve the inflow conditions. Thereupon a second connection in Mexican territory was made between river and canal. This was about 4 miles south of the boundary line. It is reported to have been first put into use on October 26th, 1904.

Neither of these connections between river and canal, commonly referred to as the Upper and Lower Mexican Headings, was provided with regulating gates. Both were already in use before, on August 28th, 1905, the canal plans were approved by the Mexican authorities. Both were simple dredger cuts of moderate depth and width. The Upper Mexican Heading, like the original canal head in California, showed a pronounced tendency to silt up. The Lower Mexican Heading, on the contrary, which came into use as a channel from 40 to 50 ft. wide and from 6 to 8 ft. deep, had sufficient fall from river to canal (reported at 1.5 ft. in about ½ mile) to give the water a scouring velocity. The bed of this heading was enlarged by the flow through it, but this, at first, caused no alarm. Large canal capacity was desirable, and natural erosion of the channel would reduce the ultimate cost of construction; but soon a time came when the progress of this channel enlargement was too rapid, and measures had to be taken to shut off the flow at the intake. The first proposition seems to have been to place a controlling gate in the canal just below the point where the Lower Mexican Heading joined the canal. The spring rise of 1905, however, came early, and this idea was abandoned as impracticable. Effort was then made to close the heading at the river. Piles were driven, brush and sand bags were used in large quantities, but as rapidly as the structure was pushed from the north side of the heading toward its south side the south bank was cut away by the river's swirling waters. Meanwhile, the river had attained its high stage (June, 1905), and the canal company was in the throes of a reorganization. The first attempt to close the heading

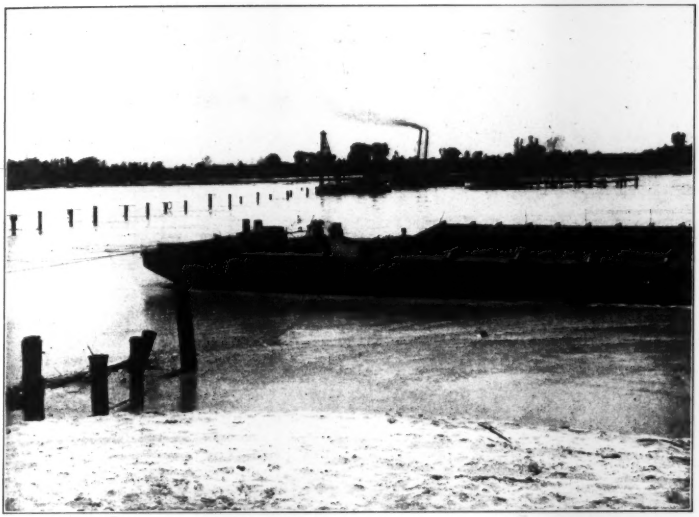


FIG. 1.—VIEW ACROSS ENLARGED LOWER MEXICAN HEADING OF IMPERIAL CANAL, AUGUST 26TH, 1906.

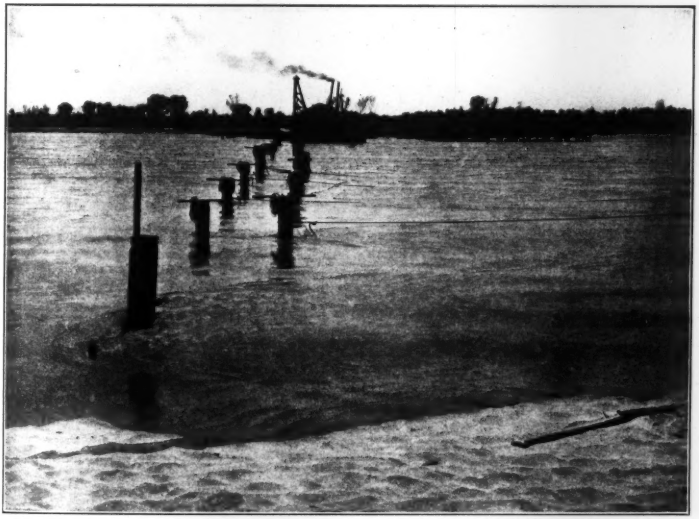


FIG. 2.—PILING FOR RAILROAD TRACK ACROSS ENLARGED LOWER MEXICAN HEADING, IMPERIAL CANAL, AT SITE OF PROPOSED DAM, AUGUST 26TH, 1906.

was abandoned. It was at this time that the direction of the work relating to the checking of the excessive flow into the canal was assumed by the railroad people, the Southern Pacific Company, at about this time, having become the financial backers of the canal enterprises which were developing territory tributary to its railway system. The railroad officials, as already stated, had recognized the importance of effecting a speedy control of the flow of water into the Salton Basin, and representatives of the railroad company were at once placed in charge of the work at the river.

The situation in parts of the Salton Basin, due to the large inflow of water, which was far in excess of channel capacities, had in the meantime become serious. There was already so much water on the salt beds that the New Liverpool Salt Company was put out of business, and was making all haste to ship the last of its salt stores out of the country. The rising waters were at the base of the railroad embankment (end of June, 1905), and it was only a question of a few days before the roadbed for a length of many miles must be abandoned. Quick action, and an early check of the inflow of water was the essential requirement.

Other interests were also menaced, however, and there were other sufferers from the flood conditions in Imperial Valley.

Structures at Sharp's Heading were in danger; those lower down on the Alamo could not withstand the flood stages of the river. Lands adjoining the Alamo and New Rivers were submerged. The threatened inundation of the main body of improved lands was prevented only by strenuous work on dikes which forced the water westward into the broad flat depression along New River. Much improved land along New River was inundated.

And all the time (summer, 1905), even while the river was falling, the volume of water presented for passage into the Salton Sea kept increasing. The Lower Heading, in the latter part of June, had been widened to 100 ft. or more, with a depth of about 20 ft. It was then discharging about 7 000 to 8 000 cu. ft. per sec., and it was but a short time before this flow was doubled. It became evident early in July that, if not checked, the river would make the Lower Mexican Heading its low-water channel.

Not only did the river do this, but it increased the canal ca-

pacity to such a degree that the canal during the flood stage of November and December, 1905, became the main channel of the river.

Turning the Lower Colorado River Back into Its Proper Channel.

Soon after the failure of the first attempt by the canal companies to stop the flow into the lower heading, they made a second attempt. This was an effort to force the entire river into a channel east of an island in the Colorado River opposite the Lower Mexican Heading. This island was narrow, and about a mile long. It divided the stream into two branches, of which, before the construction of the heading, the eastern branch had the greatest depth and carried the most water. It was proposed to close the western channel at the head of the island. Here, also, piles were driven, obliquely out from the river's right or western bank toward the head of the island, and brush and sand bags were used to check the flow of water. Three times at this point the river defeated the engineers, and impressed upon them the difficulty of the problem. Thereupon what is generally referred to as the third attempt to conquer the stream was made. This was in the fall of 1905, and the point at which the attempt was made was about 1 000 ft. below the head of the island. Here the construction of a dam with a broad brush mattress foundation was undertaken. Mats of brush, having a width of 100 ft. in the direction of the stream, were placed from bank to bank across the western channel of the river. Piles were driven in three rows through these mats, which were about 18 in. thick, and more brush was woven between these piles. This work is reported to have been making fair progress, notwithstanding a stage of the river considerably higher than the ordinary stage for that time of year, when the unexpected happened. A flood occurred on the Salt and the Gila Rivers, and a tremendous volume of water was suddenly discharged into the Colorado River at Yuma. In a few days the river rose from a low stage to become a raging torrent with a maximum discharge, on November 30th, of more than 100 000 cu. ft. per sec. This water overtopped the island. It cut around the eastern end of the incomplete dam and tore away the head of

COLORADO RIVER,
AT LOWER MEXICAN HEADING
OF THE IMPERIAL CANAL.
SKETCH, JUNE 23d, 1905.

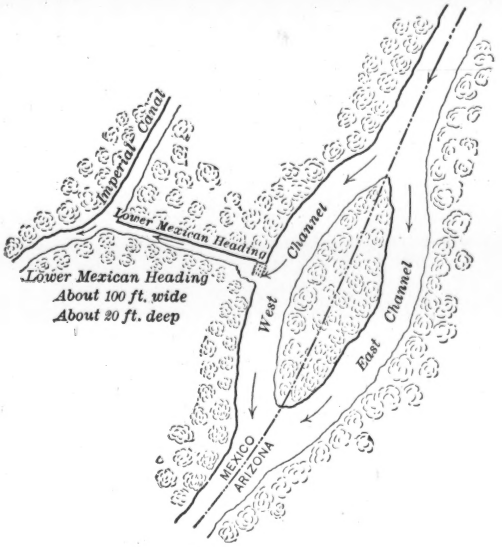


FIG. 3.

COLORADO RIVER,
AT THE LOWER MEXICAN HEADING
OF THE IMPERIAL CANAL.
SKETCH SHOWING CONDITION ON MARCH 19TH 1906

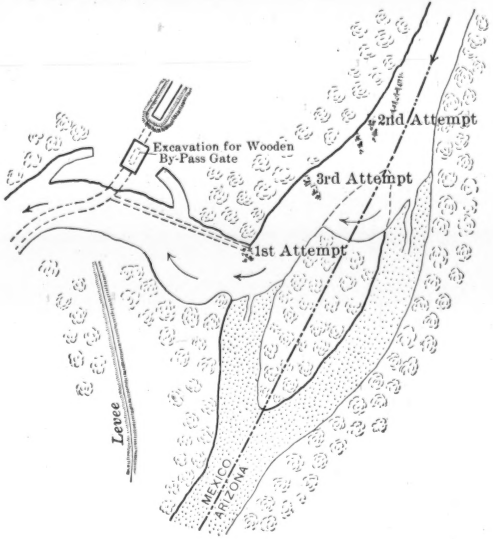


FIG. 4.

the island. It quickly cut the heading to a width of several hundred feet and gave it all the appearance of a permanent section of the main river.

This failure, so late in the fall, and the stages higher than ordinary which the river would maintain during the following winter, left the canal officials no hope of accomplishing any results without thorough preparation. It was determined, therefore, that the next attempt should be supplemented by some by-pass arrangement. A channel was to be prepared with a controlling gate through which the river, at a low stage, could be turned. Thereupon the river was to be obstructed by brushwork until its entire flow was forced through the by-pass. The brushwork dam would then receive earthwork reinforcement and be connected with levees or dikes up and down the river so that finally a closing of the controlling gates would raise the water high enough to force it to take its course, where it belonged, down the old channel of the river to the Gulf.

Two points appeared to be available in carrying out this by-pass idea, the one involving least expenditure of time and money being at the Lower Mexican Heading; the other was at the original canal heading in California. It was believed by the engineers in charge of the work that at the lower point a large gate, a timber structure, could be constructed and would be ready for use in April, 1906, when it was hoped that the stage of the river would still be low enough to justify an attempt to put a brush dam across the river. The success of this project depended upon the completion of the gate in time, and upon favorable river conditions for some weeks after its completion.

At the other point, which is several hundred feet up stream from the original canal head in California, the plans involved the construction of a large gate which it was planned should ultimately become the permanent canal headgate. From this gate an excavation of sufficient capacity to take the entire river flow at its low-water period was to be made southward, paralleling the original canal and terminating 4½ miles below in what was formerly canal, but had temporarily become river channel. As this gate was to become a permanent feature of the canal system, it was planned to be constructed of reinforced concrete.

The execution of this plan was commenced. The wooden gate

LOWER COLORADO RIVER

SKETCH ILLUSTRATING LOCATION OF DAM WHICH TURNED THE RIVER INTO ITS PROPER CHANNEL NOV. 4TH, 1906.

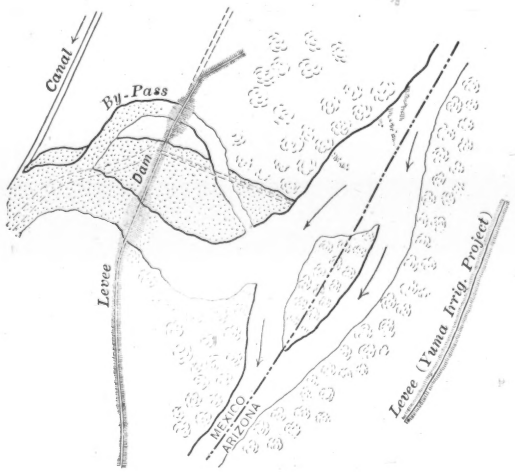


FIG. 5.

LOWER COLORADO RIVER

SKETCH SHOWING CONDITION ON DEC. 12TH, 1906.

Break in Levee occurred at 3 A.M. on Dec. 7th

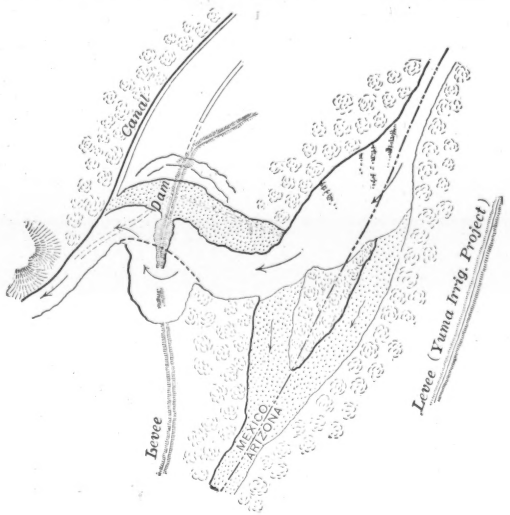


FIG. 6.

at the lower heading was constructed, but it was not ready for service as soon as had been hoped, neither did the river stages, which were too high, favor any attempt to make a closure at this point when the gate became available. The upper or concrete gate was constructed, and has been ready for use since July, 1906. Work on a canal of large capacity, beginning at the upper gate, was deferred until it should be known whether the attempt at the lower heading would succeed.

When it appeared that there was no hope of shutting off the flow into the Salton Basin in the spring of 1906, a letter was written, under date of May 10th, to the people of the Imperial Valley, by Mr. C. R. Rockwood, the Engineer in Charge, and, at that time, Assistant Manager of the California Development Company, in which he stated:

"The large wooden gate at the Lower Heading, constructed for the purpose of controlling the inflow of the Colorado through the Lower intake, is now completed and is amply able to do the work for which it is built as soon as the river falls sufficiently to allow the total flow to pass through the gates for a long enough period to construct an earthen dam across the intake. This will be when the water falls to 20 ft. on the gauge at Yuma, with a fair prospect of remaining at that height for a period of 30 days. The probability of the river doing this before the summer rise commences is so remote that, in your interest as well as that of the company, we have decided to wait until the summer flood has passed before constructing the diverting dam * * * *."

At the time this letter was written the river was already high, and kept on rising to its high summer stage, which it maintained longer than usual during 1906. In fact, throughout 1906, the river has been continually at higher stage than normal; the monthly discharge for each month is nearly if not quite equal to the maximum monthly totals ever recorded, and the aggregate discharge to date since January 1st, 1906, is already more than twice as great as the river's normal annual flow. Preparations were made for a supreme effort to stop the flow at the lower heading during the expected moderate and low stages of the river in the late fall of 1906.

A spur track about 6 miles long was constructed in August from near El Rio to the Lower Mexican Heading. A large force of men was assembled, and all the facilities at the disposal of the

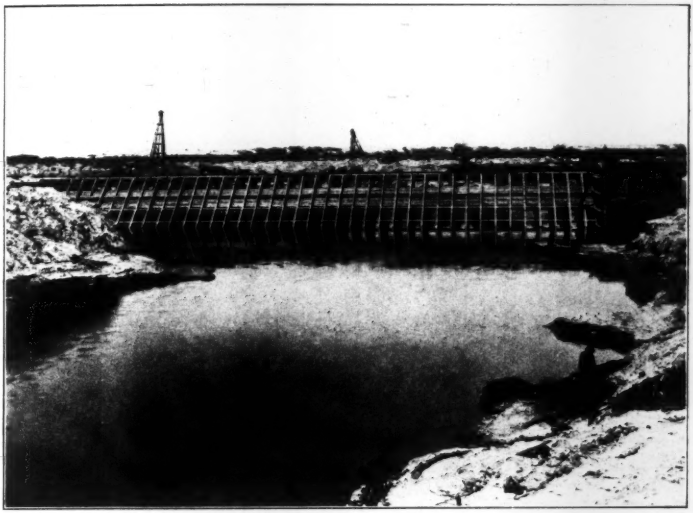


FIG. 1.—WOODEN GATE IN BY-PASS AT LOWER MEXICAN HEADING OF IMPERIAL CANAL. VIEW DOWN STREAM, AUGUST 26TH, 1906.

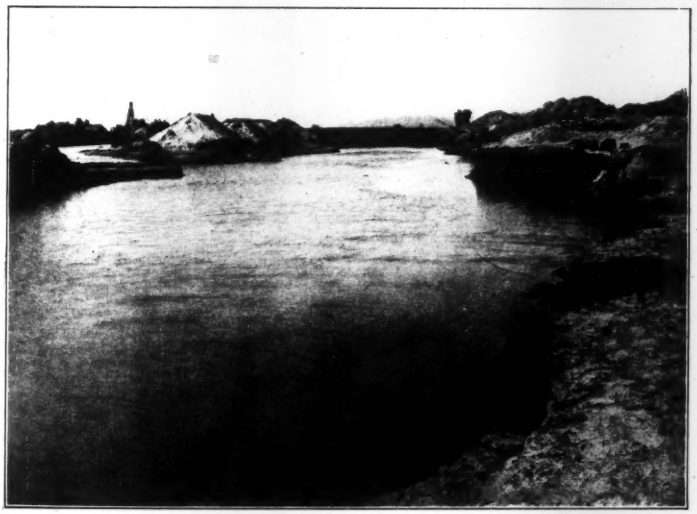


FIG. 2.—BY-PASS AT LOWER MEXICAN HEADING OF IMPERIAL CANAL, AUGUST 26TH, 1906.

railroad were called into requisition. The importance of rapid work was thoroughly appreciated.

Brush mattresses were placed successfully across the 600-ft. opening in which the river was to be held, and in which its waters were to be gradually raised by obstructing its flow with brushwork and rock. Two trestles were built across the stream, facilitating the rapid delivery of material. This work was making favorable progress, and a fair proportion of the river's flow—still far above normal—was already passing through the timber gate when, on October 11th, this gate suddenly gave way. It broke in two and its northerly half floated away. Although the situation thereupon looked hopeless, there still remained a chance of dropping rock, into the new branch of the river thus opened, faster than the water could undermine and bury it. Preparations for this work were quickly made. Orders were issued to all quarries within a radius of some 400 miles to load cars with large rock, and locomotives were pressed into service to rush the loaded cars to the work. Night and day the material thus obtained was dumped into the waterway. In the meantime the rock fill, already well advanced in the main channel of the heading, was lowered by rolling rocks from its crest, thereby increasing the flow over this barrier and reducing the flow in the by-pass channel. This operation was successful. The by-pass was completely closed on October 27th, and thereupon the dam in the main arm of the heading was raised to full height, the difficulties decreasing as the volume of flow down the river's proper channel gradually increased with the increasing height of water surface. When finally the water surface had been raised 14 ft., the entire flow of the river was in the original channel. The river was thus completely turned on November 4th. 1906

It had been foreseen by the engineers that the deterioration of the river channel below the Lower Mexican Heading, which had occurred during the year in which the river flowed inland, would result in higher water surface elevations than had theretofore prevailed. Earth dikes or levees, therefore, were extended up and down stream from the rock dam in the canal heading. These were made 10 ft. wide on top, and had side slopes of 2 to 1. They were built of material taken from borrow-pits on the canal side.

During the last days of November and early in December heavy

rains on the headwaters of Gila River sent a freshet flow down this stream, which reached its maximum at Yuma on December 7th. The water rose to 26.2 ft. on the gauge at that point (about 7 ft. above low water), and the river's flow had been increased from about 10 000 to a maximum of 56 000 cu. ft. per sec. For points near the Lower Mexican Heading this was an overbank stage. It was found that the porous material on which the levee rested would permit considerable underflow. There was a concentration of such underflow at a point about ½ mile south of the rock dam, and the water quickly cut a passage into which the levee collapsed. This occurred early in the morning of December 7th, at a time when the water stood about 3 ft. deep against the levee. The main outflow of the percolating waters is reported to have occurred in the edge of the borrow-pit which, as already stated, was on the land side of the levee, but separated from it by a berme some 40 or 50 ft. in width.

The break in the levee rapidly widened to 800 ft. The waters flowing through concentrated in the borrow-pit, flowing north toward the canal heading, and dropped into the channel from which the river had so recently been turned. The flow along the levee could not be withstood, and soon another section, some 500 ft. in length, nearer the rock dam, was cut away. Within a few days thereafter the river had cut a channel through the yielding bank deep enough to put its entire flow back upon its inland course. The work of turning the river had to be done anew, and the magnitude of the problem of holding it when once turned, is now thoroughly impressed upon the engineers who have been engaged on the work or who have followed the events on the Lower Colorado.

Effects Produced by the Inland Flow of the Lower Colorado River.

The entire flow of the Colorado is again concentrated in the watercourses which lead into Salton Basin. For a time in the spring and early summer of 1906 a small quantity of water, probably never more than 4 000 out of a maximum discharge of about 90 000 cu. ft. per sec., took the course to the Gulf by way of the river's proper channel. For some months during the high

FIG. 1.—THE BARRANCA OF NEW RIVER, ABOUT 4½ MILES NORTHEAST OF BRAWLEY. AUGUST 30TH, 1906.

FIG. 2.—THE BARRANCA OF NEW RIVER, ABOUT 5 MILES NORTHWEST OF IMPERIAL, AUGUST 31ST, 1906.

stage of 1906 there was also some flow down the Paredones, and overbank from it to the Pescadore. The aggregate quantity at any time thus reaching Hardy's Colorado, and ultimately the Gulf, by routes other than the main river was only a few thousand cubic feet per second. The remainder of the river's flow, except for the brief time that the river was turned south, went north, dropping to a plane far below sea level and traveling along lines of much steeper gradient than that of the river in a direct course to the Gulf of California.

After the flood flow of November, 1905, the Lower Mexican Heading, the canal below this point, and the Alamo River had the appearance of a continuous great river, except that for some months thereafter there was no well-defined channel through the mesquite thickets at the Lagoons. But even these thickets were broken through, the lagoons disappeared, and by August, 1906, there was a complete concentration of flow of the river, then carrying some 20 000 to 25 000 cu. ft. per sec., in one well-defined channel to within about 10 miles of Sharp's Heading where the greater part of the water took a western course through a breach in the bank of the Alamo River toward and into New River.

To illustrate the progress of the change from the river's proper course to its inland channel, it may be noted that the original lower heading, in October, 1904, had a width of from 40 to 50 ft. and a depth of from 6 to 8 ft. It had been enlarged to a width of about 100 ft. and a depth of about 20 ft. by the end of June, 1905. It had become the main channel of the river in December, 1905; was from 300 to 600 ft. wide in March, 1906, and more than ½ mile wide at some points in June, 1906. Its south bank was exposed to the violent attack of the water turned suddenly westward from a southerly course, where it left the river's proper channel, and this accounts in a measure for the rapidity with which it was cut away. Subsequent to June, 1906, during its falling stages, the river built out sand bars from the northern bank of the cut, and soon thereafter had effected the concentration of its water in a channel generally about 600 to 800 ft. wide, the south bank of which was from 3 000 to 4 000 ft. from the original north bank of the canal heading.

The large wooden by-pass gate which originally (at the begin-

ning of 1906) was within a few hundred feet (to the north) of the river's new channel was in August separated from it by broad sand bar deposits. The by-pass channel from river to gate and from gate back to river had to be made about ¾ mile long, instead of several hundred yards, as would have been the case in the early months of 1906, when the location for the gate was selected.

The immense energy of the spring flood of 1906 which poured into Salton Basin was not long in cutting great barrancas some hundreds of feet wide and from 40 to 80 ft. deep, into the alluvial soil of the Imperial Valley. The upper lip of a barranca of this kind on New River was at a point due west from Imperial on March 15th. In two months it traveled up stream some 15 miles to within a short distance of Calexico. It is reported to have progressed over portions of this distance nearly a mile in a day. Opposite Imperial, at the beginning of the year, the river was in a slight depression, practically on top of the ground, but in August it was in a chasm 80 ft. deep and upwards of 1100 ft. in width. The sides of this barranca stand vertically, with a moderate talus at points where the water has receded from their bases. The light clay loam of which they are composed breaks away as the water makes its undercuts or softens the supporting layers. The barranca, for the present, is practically impassable. The advance of its head up stream, and the direction of its travel has been a matter of grave concern to the inhabitants of the country, and particularly to those of Calexico and Mexicali. Embankments many miles in length, during the spring of 1906, had protected the main section of the improved lands between the Alamo and New River channels from inundation, and, in like manner, dikes from 4 to 6 ft. in height and 5 miles in length, kept the water out of the main portions of these two small towns. Later, the very existence of land which had before been productive farming land and of that on which the towns stand was menaced, and much of it was devoured by the river. Efforts were made to direct the line along which the deep cut should travel. To this end dynamite was used, with some success, it is claimed. The ground was shattered above the lip of the falls, and so it was attempted to make excavation easiest along the line where the least damage would result. Here and there, however, the water tumbling over the sides as well as

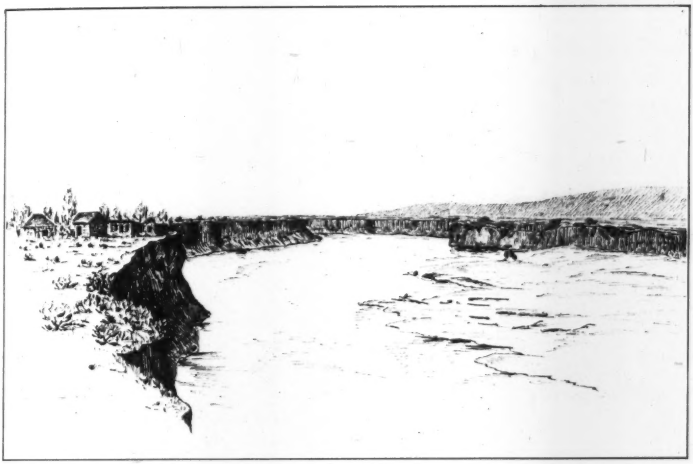


FIG. 1.—NEW RIVER, SOUTH FROM CALEXICO, AUGUST, 1906.

FIG. 2.—THE BARRANCA OF NEW RIVER AT MEXICALI, SEPTEMBER 1ST, 1906.

the upper end of the barranca has cut long and deep gullies laterally into the adjoining country. This was notably the case when the backcutting had reached a point near the five canal gates about 4 miles north of Calexico. Here a waste-water channel concentrated the overflow waters of the easterly side, and, when the deep main barranca had reached this channel, the result was a deep lateral cut which quickly undermined a canal section and deprived Brawley and vicinity of water until a by-pass could be constructed. Again, near Calexico the cutting approached dangerously close to the town. It took some of the railroad land, all the railroad improvements at Mexicali, and many of the adobe buildings of that town, and left the remainder of these towns some 40 ft. above the plane of the river's water which, but a few days before, had covered the surrounding country 4 or 5 ft. deep and had threatened destruction by inundation. It has been estimated that from 400 000 000 to 500 000 000 cu. yd. of material have been thus excavated by the river water and "ground-sluiced" into Salton Sea.

After the high-river stage of June, 1906, the rate at which the channel cutting continued decreased materially. It is reported, however, that at this time the head of the barranca is but a few miles from the point in Mexico, some 8 or 10 miles above Sharp's Heading, where the main flow of the river leaves the Alamo through breaches in its south bank on its way across country toward New River. Should this backcutting reach the Alamo, the supply of water to the canals of the Imperial Valley will be cut off, and a water famine will quickly result.

Not only has the river been carving out the great barranca above described, but, throughout the course along which the water flows from near Pilot Knob to New River, the swirling waters have deepened the channel here and widened it there until a broad continuous channel has taken the place of the original poorly defined channels through miles of mesquite thickets. The ever-increasing capacity of this new watercourse has been continually adding to the magnitude of the problem with which the railroad people have been contending. Month by month, as channel capacity has thus increased, the river has burrowed deeper into the ground at the critical point, at the Lower Mexican Heading, and thus, as time

went on, the height that water had to be raised to send it down the proper channel increased.

At the same time there was a deterioration, in some measure, of the proper river channel below the heading. The last of the water taking that course built up sand bars for some miles down stream, and these, temporarily at least, are in the way of the restored flow. The growth of weeds and willows in the river-bed while dry is a further unwelcome obstruction.

In connection with the above-noted feature of bed enlargement and deepening, it may be stated that in the spring of 1905 the water surface was already so low in the new river channel that a waste-gate in the south canal bank, some 8 miles from the Colorado, had been left high in the air. The dredger cut toward Quail River, usually referred to as Quail River Cut, which had been made in the fall of 1905 in the hope that the river might be turned toward Volcano Lake, ceased to receive water from the Alamo channel early in August, 1906. The water was too deep in the ground to continue to flow into the head of the cut. The cutting off of the discharge of the Colorado into the Gulf was complete after August, 1906.

The Canal and Water Companies.

The land irrigated by the Imperial Canal system at the time this enterprise was undertaken was for the most part government land available for entry as desert land in tracts of 320 acres, or for entry under the homestead land law in tracts of 160 acres. In either event, the use of water was essential to reclaim the land or to make it habitable. For the Imperial Valley lands no water has been available except that from the canal system controlled by the California Development Company. All desert land entries have thus become dependent upon this company and its subsidiary companies for water supply. The total area of the land filed upon in Imperial Valley is about 250 000 acres. Approximately, 90% thereof has been filed upon as desert land.

The California Development Company has been, from the outset, and still is, in complete control of all matters relating to the water supply for the Imperial Valley. It is the parent company.

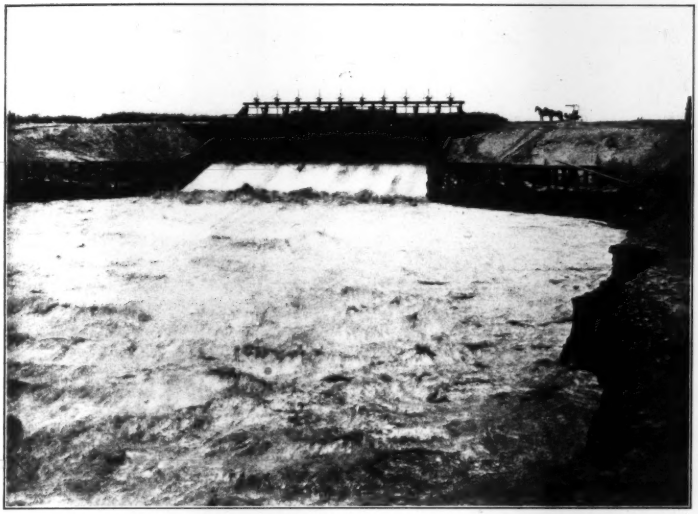


FIG. 1.—WASTE-GATE AT SHARP'S HEADING, MEXICO, ON THE ALAMO RIVER, SEPTEMBER 1ST, 1906.

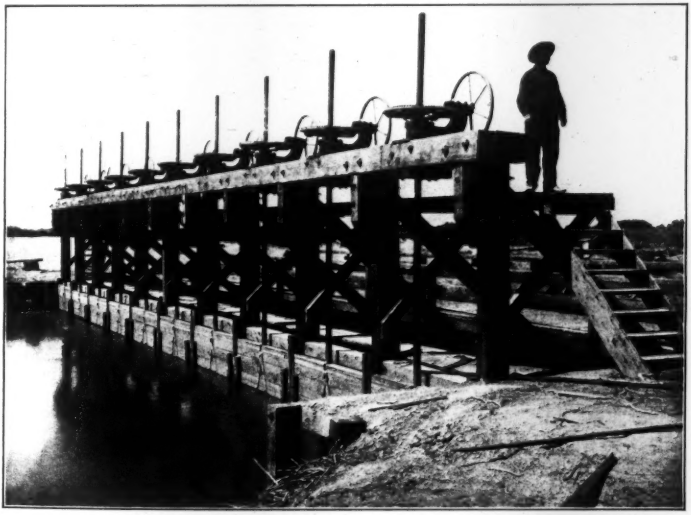


FIG. 2.—WASTE-GATE AT SHARP'S HEADING, MEXICO, ON THE ALAMO RIVER, SEPTEMBER 1ST, 1906.

All the property necessary to be controlled in Mexico by this corporation is owned by *"La Sociedad de Irrigacion y Terrenos de la Baja California"* (*Sociedad Anonym*) which, as already stated, is presumably the same corporation as the one to which the Mexican Government made the water-right concession in 1904—*"La Sociedad de Riegos y Terrenos de la Baja California"* (*Sociedad Anonym*). The California Development Company is reported to own or control all the stock in this Mexican corporation.

The desert land law required that those who perfect title to the land entered must acquire a water right. The California Development Company did not propose to make the landowners participants in the management of its affairs, neither did it undertake to issue water rights which would run as appurtenances with individually owned tracts of land; but it arranged for the organization of a number of sub-companies, separate corporations, the stockholders in which should be landowners, and in which the stockholder was entitled to a proportional use of the water controlled by the sub-company. Eight such mutual water companies were planned, and each of the eight, it was proposed, should own and manage its own canal system, receiving water from the parent company. The capital stock of each company was fixed in relation to the area to be commanded by the sub-system of canals, so that a share of stock in any sub-company would entitle its holder to substantially the same service as a share in any other company; but the water to which ownership of stock entitles its holder must be used within the territory assigned to the sub-company issuing the stock.

The holder of stock pays for the water used at the rate of 50 cents per acre-foot, paying for at least 1 acre-foot per annum on each share of stock, whether or not any water is used.

All the stock of the sub-companies (designated by number, as Imperial Water Company No. 1, etc.) passed into the possession of the California Development Company, supposedly in payment for the canal system to be built by the parent company. The water stock thus remained under control of the parent company. One of the companies, No. 8, however, bought back its entire capital stock from the Imperial Construction Company, to which it had been transferred, substituting bonds therefor to the amount of $500 000.

The relation in which the parent company stands to the mutual water companies is illustrated by the terms of an agreement made on July 24th, 1901, between the California Development Company, the Mexican Company, and Imperial Water Company No. 1, which last named is by far the most important of the several mutual companies. This agreement obligates the Mexican Company (the head of Imperial Canal No. 1 being in Mexico), upon demand of the mutual company, to furnish to the latter water as demanded not to exceed 4 acre-feet per annum for each outstanding share of stock, but conditional upon an aggregate delivery not to exceed 400 000 acre-feet per annum, and conditioned upon an annual payment of 50 cents per acre-foot.

The annual payments are to be based upon the amount of stock outstanding on the first day of June of each year, and are to be for not less than 1 acre-foot of water for each share. The price of 50 cents per acre-foot is under no circumstances to be increased. The California Development Company agrees to construct and maintain a canal, from the international boundary line to the laterals of the stockholders, of sufficient capacity to supply 4 acre-feet of water per annum for each outstanding share of stock. It also agrees to construct a system of distributing ditches for the delivery of water to a point on each quarter section of land from which it is practicable to irrigate the same by gravity flow. All ditches and laterals are to be turned over to the mutual company. The Mexican Company is to deliver 2% more water at the boundary than is required to supply the demand of the mutual company, in order to cover seepage and evaporation losses. The canal system is to be kept in good repair by the California Development Company, but this company does not hold itself responsible for failure to deliver water from any cause beyond its control. Finally, the California Development Company and the Mexican Company each guarantee the perpetual performance of all the terms and conditions of the agreement by the other. The entire control and management of the canal system, including the work in Mexico, is thus turned over to the California Development Company.

Six other mutual water companies have made similar agreements with the California Development Company and the Mexican Company.

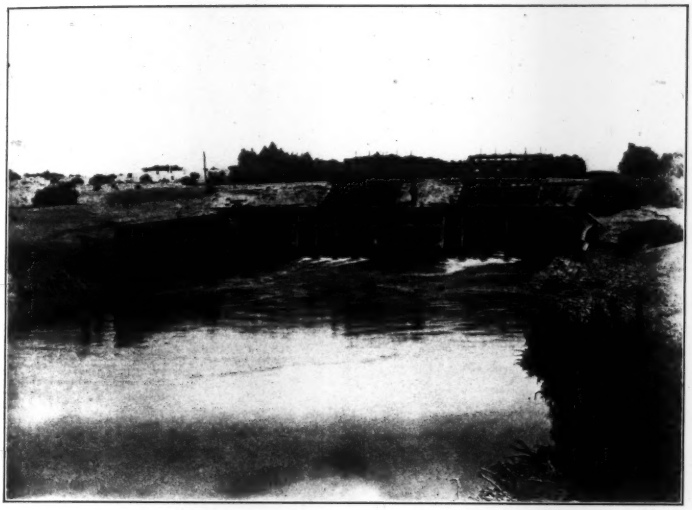


FIG. 1.—HEAD-GATE OF CANAL OF IMPERIAL WATER COMPANY NO. 1, AT SHARP'S HEADING, MEXICO, SEPTEMBER 1ST, 1906.

FIG. 2.—BRUSH MAT ON SOUTH SIDE OF LOWER MEXICAN HEADING, IMPERIAL CANAL. LOOKING NORTH. AUGUST 26TH, 1906.

Imperial Water Company No. 1 was organized to distribute water to 100 000 acres of land lying between Alamo and New Rivers and extending northward from the international boundary line.

Imperial Water Company No. 2 was organized for the irrigation of a small area between the same rivers, beginning 25 miles north from the international boundary line.

Imperial Water Company No. 3 was merged with No. 1.

Imperial Water Company No. 4 was organized for the irrigation of 17 500 acres lying between Districts Nos. 1 and 2.

Imperial Water Company No. 5 was organized originally for the irrigation of 100 000 acres, but this was subsequently reduced to 50 000 acres east of Alamo River, commencing at a point about 7 miles north from the international boundary line.

Imperial Water Company No. 6 was organized for the irrigation of 25 000 acres just north of the boundary line and west of New River.

Imperial Water Company No. 7 was organized for the irrigation of 20 000 acres just north of the boundary line and east of Alamo River.

Imperial Water Company No. 8 was organized for the irrigation of 40 000 acres west of New River and about opposite the territory assigned to No. 4.

The bonds, in the amount of $500 000, issued by Imperial Water Company No. 8 are 20-year bonds bearing interest at 6 per cent. The irrigators, in addition to paying 50 cents per acre-foot of water, also pay for the maintenance of the canals and for their operation.

The Imperial Land Company is another corporation which was apparently originally closely related to the California Development Company. It was entrusted by the latter with the advertisement of the enterprise, the locating of settlers upon the land, and with the sale of the stock issued by the above-mentioned mutual companies, and was paid a commission of 25% on all sales of stock.

When, finally, the parent company became too deeply involved, financially, some of its principal stockholders organized the "Delta Investment Company," to which, it is understood, certain securities were transferred.

These matters are touched upon in order to give some idea of the involved conditions relating to the control of the water for the supply of Imperial Valley, and to show how inadequately the landowners who have bought water have been protected.

At the present time the Southern Pacific Company is, as already explained, directing the affairs of the California Development Company and of its affiliated concern, the Mexican Company, acting in this matter virtually as a trustee. The railroad company saw the possibilities of the valley and the urgent need for aid. Except for energetic measures, made possible by this outside financial aid, the turning of the Colorado River back into the Gulf of California by the canal company would have been hopeless from the start. With the aid of the railroad, and under the direction of its engineers, there was hope of success.

The hundreds of thousands of dollars which have been advanced to the canal companies for expenditure in Mexico were put into the work by the railroad company at more than ordinary risk. There was much at stake, and the protection of some of their own property was involved. Whether the canal companies can ever repay the money thus advanced, whether Mexico can be held responsible and will make good the losses that have resulted from work done under Mexican concessions, upon Mexican territory, by a Mexican corporation, are questions which the railroad company has probably not yet considered. To stop the river first, and avert the greater calamity which was threatened, is the principle on which the railroad officials have been acting.

Development of Resources in the Imperial Valley.

Under the influence of irrigation, the Imperial country made rapid progress in all the industries common to good farming districts. The irrigated area expanded quickly to about 80 000 acres, and a much larger area was under ditch. A branch railroad, nearly 40 miles in length, was constructed by the Southern Pacific Company southward from Imperial Junction to the international boundary at Calexico, and the extension of this road was well advanced into Mexico before sections of it were destroyed by New River.

It appears probable that about 275 000 acres of land in the Im-

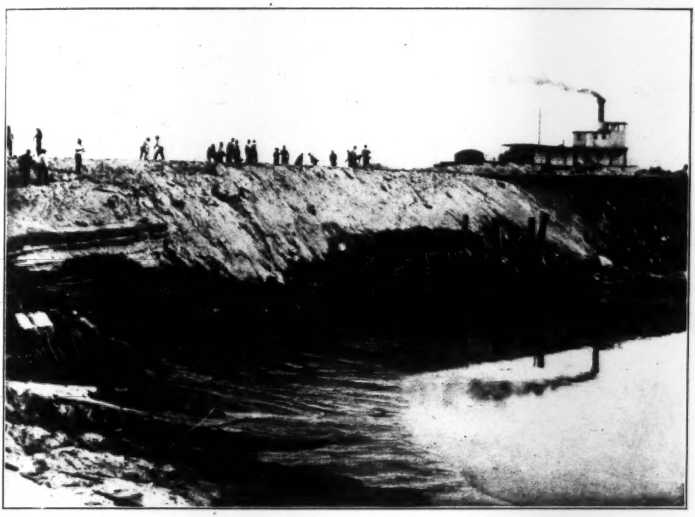


FIG. 1.—DAM ACROSS LOWER MEXICAN HEADING, IMPERIAL CANAL, NEARING COMPLETION, NOVEMBER 13TH, 1906.

FIG. 2.—BY-PASS AT LOWER MEXICAN HEADING, IMPERIAL CANAL, CLOSED NOVEMBER 13TH, 1906.

perial country are suitable for irrigation. The conditions, however, are such that the system of works supplying water to this region should be absolutely reliable. The region has no auxiliary sources with which to safeguard a supply from the Colorado River. There is no soil water within reach. All water, even that for domestic use, must be brought into the country by the canal system.

Some idea of the progress which has been made in Imperial Valley is conveyed in the following statement from a letter written by Mr. W. F. Holt, President of the Imperial Water Users' Association, under date of January 17th, 1905.

"This country is practically only 3 years old, still, to-day it has about 10 000 population; it has all modern improvements, such as telephone systems, electric light plants, ice plants, water-works systems, dozens of commercial houses, railroads, good hotels, and all other things that go to make up a prosperous community. There are now in the valley over 20 000 cattle, over 30 000 hogs, and all other live stock, poultry, etc., running into thousands. The grain men estimate that the present crop will amount to over $1 000 000. There are eight public school districts in the valley to-day, employing from one to four teachers each. There are six post offices. * * * * There are two banks in the valley, both doing a large business, and two other banks are now erecting buildings. * * * The Imperial station during 1904 was the third station in amount of business done on the Southern Pacific System in Southern California, Los Angeles being the first and San Pedro the second."

Irrigation in Lower California, Mexico.

Some of the water diverted by the California Development Company from the Colorado River has been used on lands in Lower California. There has not been any subdivision of land there, nor any attempt to colonize. Irrigation has been confined to the operations of the California and Mexico Land and Cattle Company, which has had under cultivation by irrigation some 4 000 acres near Mexicali, and is prepared to extend this area rapidly to cover entirely a tract of 10 000 acres, as soon as the supply of water is sufficiently safeguarded.

The ownership of this 10 000-acre tract is apparently in the Colorado River Land Company; it was acquired, with rights to water, from the California Development Company or its affiliated Mexican

corporation. The Colorado River Land Company, it is understood, now owns 450 000 acres acquired from Guillermo Andrade, and holds a lease and contract of sale to 223 560 acres more. For all this land, which has passed through the hands of Andrade, water rights in the Imperial canal are claimed, and it is asserted, moreover, that the contracts for water for this land were made with the canal company before the California Development Company had made its contracts with the mutual water companies of the Imperial Valley. It is well known that much of this land for which rights to water from the Imperial Canal are claimed is located favorably, and is otherwise well suited for irrigation.

Flow of Colorado River, and Time Required to Fill Salton Basin.

The Colorado River system drains portions of Wyoming, Utah, Colorado, and New Mexico. It has a water-shed of 225 000 sq. miles. The normal annual run-off from this area is approximately 9 000 000 acre-feet. This is distributed in the several months about as shown in Table 1.

TABLE 1.

Month.	Second-feet.	Acre-feet.
January	4 500	280 000
February	4 800	270 000
March	5 100	320 000
April	9 500	570 000
May	29 000	1 800 000
June	40 000	2 400 000
July	23 000	1 400 000
August	9 000	550 000
September	6 700	400 000
October	5 500	340 000
November	5 200	310 000
December	4 700	290 000

The gauge records prior to 1894 were not used in the preparation of Table 1, owing to the fact that the relation between water-surface elevation and discharge for earlier years has not been determined.

The flow of 1906 has not been considered in preparing Table 1. The run-off for 1906 will probably exceed that of 1905. With 1906 included in the computation, the river's normal flow at Yuma should be noted at about 10 000 000 acre-feet per annum.

It seems probable that the normal flow of the river, if estimated with the two years of excessive run-off, 1905 and 1906, included, would be entered at too high a figure. It has not seemed proper to include both these years (the latter still incomplete), in view of the fact that it is known from gauge records at Yuma since 1878 that the discharge in no other year exceeded much more than one-half of the discharge in either of these two years. The gauge records indicate that the occurrence of years of such great flow is not frequent.

The area of Salton Basin below a contour line 30 ft. above sea level is 1 947 sq. miles. As the water rises from the bed of the basin to the height of a full basin, the area of Salton Sea will increase, as follows:

For	water	surface	278	ft.	below	sea	level:	Area	0	sq.	miles.
"	"	"	253*	"	"	"	"	"	247	"	"
"	"	"	200	"	"	"	"	"	447	"	"
"	"	"	150	"	"	"	"	"	647	"	"
"	"	"	100	"	"	"	"	"	916	"	"
"	"	"	50	"	"	"	"	"	1 146	"	"
"	"	"	at sea level					"	1 753	"	"
"	"	"	30	ft.	above	seal	level:	"	1 947	"	"

The rate of evaporation from a large open water surface in the Colorado Desert has not been determined. Some experiments made at Yuma by the Agricultural Department indicate that the evaporation will be nearly 7 ft. per year from small confined bodies of water directly exposed to the full energy of the sun's rays and freely wind swept. It is believed that the evaporation from a large body of water will be less than this. Evaporation near Tulare Lake in San Joaquin Valley, is about 4 ft. per annum.† The climate there is not as continuously hot and dry during the summer as in the Colorado Desert. It is to be assumed that evaporation in the desert will be considerably in excess of that near Tulare Lake, and, in the absence of better information, it may be placed at about 6 ft. per annum effective (not allowing for rainfall, which is probably about 3 in. per annum). On this assumption, and on the further assumption that the river's annual output will be 9 000 000 acre-feet of water, it is possible to forecast what will probably be the

* Water surface on January 1st, 1906.

† Physical Data and Statistics—State Engineering Dept., California, p. 380.

stage of the Salton Sea, if the Colorado River continues to flow inland. The actual stage will vary from this forecast according to the variation of the annual discharge from the assumed normal river flow. It will rise more slowly if there be a succession of dry seasons, and more quickly if the conditions of excessive rain of the last two years continue to prevail. It will also vary from this forecast if there is material error in the assumed rate of evaporation.

Evaporation from the Salton Sea at 6 ft. per annum plus rainfall would reduce the water accession as follows:

At water surface 200 ft. below sea level, 1 700 000 acre-feet per annum.
At water surface 150 ft. below sea level, 2 500 000 acre-feet per annum.
At water surface 100 ft. below sea level, 3 500 000 acre-feet per annum.
At water surface 50 ft. below sea level, 4 400 000 acre-feet per annum.
At water surface, sea level, 6 700 000 acre-feet per annum.
At water surface, 30 ft. above sea level, 7 400 000 acre-feet per annum.

The probable annual increase in the quantity of water in the Salton Sea, therefore, will be (river discharge minus evaporation):

At water surface			200	ft.	below	sea	level,	7 300 000	acre-feet.
"	"	"	150	"	"	"	"	6 500 000	"
"	"	"	100	"	"	"	"	5 500 000	"
"	"	"	50	"	"	"	"	4 600 000	"
"	"	"	sea level,					2 300 000	"
"	"	"	30 ft. above sea level,					1 600 000	"

The water surface, therefore, must be expected to rise at the following rates per annum:

At water surface			200	ft.	below	sea	level,	25.5 ft.
"	"	"	150	"	"	"	"	15.7 "
"	"	"	100	"	"	"	"	9.4 "
"	"	"	50	"	"	"	"	6.3 "
"	"	"	sea level,					2.0 "
"	"	"	30 ft. above sea level,					1.2 "

At the close of 1906 the Salton Sea will be at an elevation of about 206 ft. below sea level. Applying the above-noted annual probable rates at which the water will rise, it will be found that the following stages of the water appear probable:

January	1st,	1907,	206	ft.	below	sea	level.
"	"	1908,	182	"	"	"	"
"	"	1909,	163	"	"	"	"
"	"	1910,	147	"	"	"	"
"	"	1912,	120	"	"	"	"
"	"	1914,	99	"	"	"	"
"	"	1916,	82	"	"	"	"
"	"	1918,	67	"	"	"	"
"	"	1920,	53	"	"	"	"
"	"	1925,	26	"	"	"	"
"	"	1930,	9	"	"	"	"
"	"	1935,	2	"	above	"	"
"	"	1940,	12	"	"	"	"
"	"	1945,	20	"	"	"	"
"	"	1950,	27	"	"	"	"

At a lake stage of 23 ft. or more above sea level, there would be some outflow through New River and Hardy's Colorado to the Gulf, and thereafter the rise to higher lake stages would be very slow.

An assumed mean annual delivery of 10 000 000 acre-feet of water into the Salton Sea would have made the following water stages appear probable:

January	1st,	1908,	179	ft.	below	sea	level.
"	"	1910,	140	"	"	"	"
"	"	1915,	80	"	"	"	"
"	"	1920,	40	"	"	"	"
"	"	1925,	14	"	"	"	"
"	"	1930,	3	"	above	"	"
"	"	1935,	16	"	"	"	"
"	"	1940,	27	"	"	"	"

EFFECT OF A PERMANENT RIVER CHANGE ON IRRIGATION POSSIBILITIES.

The seriousness of the situation on the Colorado River will appear from some additional considerations.

During all the time required to fill Salton Basin, the Colorado River will have gradients at points below Yuma exceeding those which it has long maintained with but little and very slow change while it flowed to the Gulf. The river, therefore, if the flow to the Salton Basin be not stopped, will cut its channel deeper into the ground near Yuma. Its stages will be lower there than in the past for corresponding volumes of flow.

Depression of the river channel will continue during the years that the water surface of Salton Sea remains materially below sea level; but, as the water of the sea rises to sea level and to higher elevations, there will be a corresponding rise in the bed of the river channel and a gradual restoration of channel elevations at and above Yuma to those which have been maintained so long in the past.

No prediction can be made as to the amount of channel deepening which will result from a continued flow into Salton Basin. This will depend in large measure upon the character of the material encountered as the river-bed sinks deeper below the present surface of the alluvial deposits at and above Yuma. It is quite possible that at points below Yuma, where light alluvial material to a great depth is to be assumed, the cutting will approach or even exceed 100 ft. in depth.

Whatever this deepening of channel at Yuma may be, it will be a serious matter for the lands of the river valleys all the way up stream to the river's cañon. Throughout all these valleys the water of the river may cut its channel so deep that irrigation will be out of the question, and it may swing back and forth until it cuts away most of the land of these valleys now awaiting the application of water.

The future of some 400 000 acres of land along the Colorado River is thus imperiled. It is not probable that any reasonable project can be devised for their protection or for their irrigation, if the river be not speedily turned, until the Salton Sea approaches its full stage.

If the river be not turned during the present low-water period (before the end of March, 1907), then the succeeding high stage may menace the work now being done by the United States at Laguna, some 12 miles above Yuma. The construction at that point of a

dam resting upon river silt may have to be abandoned, and all irrigation projects along the lower river may have to be modified to suit new conditions, or be entirely abandoned.

During the time that the river discharges into Salton Sea it will (as already stated) drop most of its silt at the lake margin. It will refill the great New River barranca. Progressively, as the water of the inland lake rises, it will build up its bed again, until the water stages at Yuma are again as high as they have been heretofore. When at last Salton Basin is full, the river will not only be again at and above bank heights at its high stages, but the gradients along its course inland will become less than the gradients in the direction toward the Gulf. The river will then be ready to resume its flow to the south, and the turning can probably be effected easily across the dry, barren waste which will have taken the place of the present well-watered river delta.

Operations of a Private Corporation in Mexico Caused the Temporary Inland Flow of the River.

The artificial cut in which water flows westward from the Colorado to the Imperial Canal, which has become the river channel, is located in Mexico, as already explained, 4 miles below the boundary line. It was constructed by a private canal company operating under a concession from Mexico. The work was at no time under the supervision or control of any branch of the Government of the United States. It came under the observation of officials of the Department of the Interior, who, in their work on the reclamation projects of the Lower Colorado, and their studies of the duty of water in the Imperial region, measured the stream flow at Yuma and the canal flow at a number of convenient points. As soon as the flow of the water on the Lower Mexican Heading exceeded irrigation requirements, and attempts were made to stop the flow from the river into this heading, the representatives of the Reclamation Service who had charge of this work foresaw the possible effect of a large diversion at that point, and made frequent reports on the situation. In this way the Department of the Interior has had knowledge of the operations on the river, but neither at the river nor in the Imperial Valley has it, or any other branch of the Federal Government, been connected in any way with the enterprise that

has led up to the present situation. This statement would be superfluous, except for the fact that in many sections the impression prevails that the irrigation work in the Imperial Valley had been undertaken or taken over by the Reclamation Service.

It should be noted that a proposition to assist the Imperial Valley was under consideration by the Reclamation Service in 1904. At that time a Water Users' Association had been formed, and, after negotiations with the owners of the Imperial Canal System, this Association urged upon the Reclamation Service the purchase of the properties of the California Development Company with certain exceptions, for the sum of $3 000 000. This price was considered too high. This and other circumstances, notably the complications resulting from the necessity of carrying water through Mexico, made it impracticable for the Reclamation Service to come to the aid of the settlers.

Colorado River as an International Boundary. The River's Navigability.

The Colorado River, beginning at a point about 8 miles below Yuma, where the boundary line between California and Lower California leaves the river, is the boundary between the United States and Mexico for a distance of about 20 miles. The position of this part of the international boundary is defined as the middle of the stream. The navigability of this part of the river was apparently recognized in the treaties of Mexico relating to the international boundary. According to the provisions of the treaty of Guadalupe Hidalgo (1848), neither country is to permit works throughout the portion of the river which is common boundary which would interfere with the navigability of the river. The treaty of 1853 cancels this provision, but guarantees to the United States the free and uninterrupted passage of vessels and citizens so far as the river forms the common boundary between the two countries.

There has been practically no commerce on the river below Yuma since the completion of the Southern Pacific Railroad in 1877; and there has been no expenditure by the War Department for its improvement. The result of a preliminary examination of the Colorado River from Yuma to the Mexican boundary line is reported as

follows by General G. L. Gillespie, Chief of Engineers, under date of December 1st, 1903:

"In the opinion of Captain Jadwin, the Division Engineer, Lieut.-Col. D. P. Heap, Corps of Engineers, and the Board of Engineers for Rivers and Harbors, which has reviewed this report, in pursuance of Sections 3 and 14 of the above mentioned act, the locality is not worthy of improvement by the United States."

Referring to the situation at the Lower Mexican Heading, Col. W. H. Heuer, Engineer Corps, U. S. Army, under date of September 13th, 1905, reported:

"Under these circumstances it would seem that the costs of investigation, plans, estimates and work of closing or controlling the outlets, if such should be required, should be borne not by the War Department, but by others whose interests involved are incomparably greater."

DISCUSSION.

Mr. Price. W. G. PRICE, M. AM. SOC. C. E. (by letter).—The writer wishes to make the following suggestions as to a form of dam which might be used to close the outlet of the Colorado into the Salton Basin.

Owing to the rapidity with which water sometimes cuts the silt deposit from under a dam, it is not safe to build such dams with piling or with piling driven through a single thick mattress. The cutting away of the silt below the mattress will usually undermine the piling and cause that part of the dam which is supported by piling to be carried away. The dam must be built so that the water cannot move any part of it in any direction except vertically downward. Such a dam must necessarily be built wholly of mattress and ballasted with rock, one mattress on top of another being added as the structure settles, until the water can move it no farther, after which it can be carried to the desired height. By carrying the mattress structure far enough into and on top of the river banks to prevent the water from cutting around it, the work can be completed as rapidly as may be convenient, and without fear of failure.

For closing a channel of a river of the size and velocity of the Colorado, the mattress should be not less than 2½ ft. thick, and should consist of alternate layers of brush, the layers being laid at right angles to each other and with such distribution as to form pockets about 1½ ft. deep to hold the rock ballast. The mattress must be bound together so that the water cannot cause it to fray out at the down-stream edge and go to pieces stick by stick; and, where the structure is to be permanent, metal must not be used for a permanent fastening of the parts. All fastenings must be of wood, which will not decay under water; metal fastenings will be cut away by corrosion and by the impact of sand particles. No. 10 steel wires laid on the bottom of the Upper Mississippi River by the writer were reduced in sectional area about one-half in 10 weeks by water-driven sand striking them. The wires were taken up and examined every day. They were always bright, and showed no signs of corrosion. The velocity of the water was seldom as great as 4 ft. per sec.

In building the sill dams in the Atchafalaya River, in 1887, to limit its outlet capacity from the Mississippi River, the writer used a crib form of willow mattress. The Atchafalaya at low water usually carries all the Red River flow, and at flood time its discharge has been more than 400 000 cu. ft. per sec., with a very high velocity.

The foundation for these dams was mostly white sea sand about

80 ft. in depth. The foundation mattress was 304 ft. long, parallel to the river, 3 ft. thick, and each successive mattress had a shorter length. The dams settled many feet as they were being built up, but they stopped settling before they were finished, and they have held against the floods of the Red and Mississippi now for nineteen years. Mr. Price.

The writer built one dam, for closing a chute in the Missouri River, on a foundation of very fine sand, and one small part of it settled about 18 ft. before the layers of mattress could be carried above the water surface. A dam built in this way will at first permit much water to flow through it, but dead leaves and other débris, carried by the river at flood time near the bottom, will rapidly choke up the openings between the willows.

RUTGER B. GREEN, M. AM. SOC. C. E. (by letter).—This paper is a very valuable contribution, as it gives an opportunity to witness more geology in a year than usually occurs in an epoch. It is to be hoped that the author will contribute other papers of this kind, for when the immensity of the general situation is considered, it seems probable that the future may direct less scrutiny to the builders of the Imperial Canal and more to the whole problem. Mr. Green.

If there were records showing, for centuries back, the efforts of the great floods of this water-shed to find an outlet through the silt-choking mouth of the Colorado River, it is possible that they might show that the Imperial Canal merely pricked a bubble that was ripe to burst, and it is well that the present difficulty comes now as a warning instead of years hence when the damage might be a hundred-fold greater. Geology shows that at least once before our historic period the Colorado River may have so choked with silt its outlet at the sea that its floods, backing up under the increased friction, filled this Salton Sink 20 ft. above present sea level, and then under the increased head flushed out their outlet to the Gulf of California. Apparently, this present Colorado Desert may have acted as an automatic flush-tank on a by-pass with silt valves.

That there have been occasional overflows of the flood waters of the river into this basin in very recent years would indicate a possibly rising river level and gradually approaching danger. Mr. Grunsky states that small channels, cut through the river's bank by recent floods, have subsequently choked themselves by depositing driftwood and silt against the mass of vegetation that springs up in the channels. Admitting that Nature has apparently done an immense amount of very permanent construction in just this frail temporary-looking style, still, considering the power of the floods and the quantity of silt they bring down to choke their sea

Mr. Green. outlet, it does not seem impossible that in this particular case her work was not intended to be permanent, and that the present trouble would have come merely a little later if the Imperial Canal had not been cut.

It is to be hoped that the magnitude of the interests at stake will induce the United States Government, either with or without Mexican aid, to make systematic records and studies of the whole situation, so that the engineers of the future may have suitable information to guide them in the work that evidently lies waiting for them.

Mr. Dabney. T. G. DABNEY, M. AM. SOC. C. E. (by letter).—This paper is extremely interesting and instructive. The complete manner in which the entire subject has been presented is evidence that the author was thoroughly equipped for the task.

It appears that, on November 4th, 1906, after most heroic efforts, the work of closing the outflow from the Colorado River was accomplished, which, in view of the difficulties encountered, was a most notable achievement, reflecting great credit on the engineer who directed the work.

Some brief reflections are here offered on the cause of the failure of the enterprise. The most impressive feature is the apparent insignificance of the agency which caused such a great disaster, reminding one of the fable in which the mouse gnawed through the cords which bound the lion.

It would seem that the integrity of the main structure was dependent on the maintenance of a comparatively inconsiderable levee, extending above and below the dam across the diversion channel. The height of the levee is not stated, but it may be inferred that it was not great. The crown width was 10 ft., with side slopes of 1 on 2. On the Lower Mississippi River the minimum levee section has a crown width of 8 ft., with slopes of 1 on 3.

The failure of the levee was evidently not due to weakness of structure, as the depth of water against the embankment was only 3 ft. The disaster was caused by weakness of foundation, which was emphasized by a borrow-pit on the "land side" of the levee. The depth of the excavation is not stated.

It appears that a grave error was made in excavating for material on the "land side" instead of on the "river side" of the levee, an error which, doubtless, was recognized by the engineer-in-charge after the break.

There is no allusion to any excavation for foundation under the levee, and, from the writer's observations of methods of levee building in California, it is probable that this precaution was omitted. In the writer's practice in levee building, such an excavation is deemed indispensable. It is called, in the vernacular, a

"muck ditch," and its size varies somewhat according to conditions, but the standard section is: top width, 12 ft., bottom width, 8 ft., depth, 7.5 ft. The excavation and refilling are done with teams and scrapers, the material for refilling being selected where practicable. Had such a "muck ditch" been excavated and carefully refilled, and had the borrow-pit been located on the "river side," is it not probable that this great disaster would have been averted? Mr. Dabney.

There is no information as to whether remedial measures were applied after the weakness of the levee foundation developed, or whether there was sufficient time for such application. Under similar circumstances, along the Mississippi levees, the readiest and most obvious method of combating such danger is by ponding water, over the orifice of the underground flow, by a ring of earth-filled sacks surrounding the escaping water. In this way the velocity head is reduced, and the displaced material is prevented from being washed away. If sacks are not at hand, a "hoop" of earth is built, with a spillway of boards.

If the defective region is too extended for this treatment, a dam, or "sub-levee," may be thrown across the borrow-pit on either side of the dangerous locality, with perhaps a "curtain" in the rear of the pit, connecting the cross-levees in order to increase the depth of the ponded water.

Of course, in the case under discussion it is conceivable that the action of the destructive agencies may have been too precipitate to admit of any remedial measure being taken, and it seems probable that this was true.

W. DE H. WASHINGTON, ASSOC. M. AM. SOC. C. E.—The speaker recently visited the scene of the break in the Colorado River, and will attempt to describe briefly some of the events which have occurred since the last break described in Mr. Grunsky's able paper, although no adequate idea of the magnitude of the problem can be obtained from any description. Mr. Washington.

At that time the break was nearly half a mile wide, the velocity of the water in the river was from 3 to 6 miles per hour, and the channels were from 20 to 35 ft. deep.

The material in the river bottom has little or no value as a holding agent. One of the discussors refers to a method of handling materials "as fine as sea sand," but, in comparison with the Colorado River silt, sea sand is almost as coarse as cobble-stones. This silt, when wetted and rubbed between the fingers, gives no feeling of grit, whatever; neither can one hear any sound of grit when holding it close to the ear, and rubbing it in the fingers.

During low water, when the river bottom is dry, one may stand on this material, and it seems to be quite firm, yet, by pressing the foot upon it several times the sensation is that of standing upon a

Mr. Washington.

mattress resting upon water. It feels as though that slight pressure of the foot had made the soil beneath as fluid as water. The surface or top covering, however, may be removed, and no water will be seen, only wet mud which, apparently, has not enough sustaining power to support one's finger.

The silt in the river bottom, when dry, almost invariably cracks in the form of pentagonal prisms. From above, its appearance is not unlike the peculiar formation at the Giant's Causeway. The cracks are so large that one's hand may be thrust into them, and so deep that one cannot see the bottom. It might be thought that one could walk and stand upon this cracked and baked soil, but it is treacherous; the speaker tried it, and sank almost to the hips. The surface may be dry for a foot or two, but there is no sustaining power below that. This material will not expand again if water is placed on it, but will simply remain in that form, the angular parts only melting and filling the interstices. It has very little consistency, and if any water comes in contact with it (even the 3 ft. of head mentioned in the paper), it will find its way through it, so that unless the silt is entirely removed from under a levee before its construction, the water is likely to wash it out. Without doubt, this material is composed of exceedingly fine alkali dust carried down by the river from that vast region between the Sierra Nevadas and the Rockies. The Colorado is very seldom clear, but carries from 2 to as much as 7% of suspended matter, and the fact that such a large percentage of this material is carried in suspension for so great a distance gives some idea of its extreme fineness.

Like the Mississippi and the Nile, the Colorado has built up its bed until it is practically running in a channel above the level of the surrounding country on both sides; consequently, any break in the bank gives it an opportunity to rush away with a rapid current. Breaks in the Mississippi levees are readily closed because the slope of the land away from the river is not great enough to cause much current across the country; and there is no channel cutting, as the bottoms are usually level, and the water spreads and soon begins to back up.

Away from the banks of the Colorado, however, there is a slope of about 5 ft. per mile, and the scouring action of such a river in such material is very great. Down in the desert a fall of about 70 ft. has been formed by the back-cut of the river; in fact, it has back-cut in the same way as the Niagara. There is little doubt that Niagara Falls were at one time as far down as Lewiston, at the foot of the gorge. This can easily be understood by examining the gorge; and it is known that, from year to year, the falls are gradually receding toward Buffalo.

Mr. Washington.

When the total flow of the Colorado was diverted from the old into the new channel, the old channel was left dry. The greatest river flow is about 120 000 cu. ft. per sec., and this was the volume of water with which the engineers had to cope. The rises in the river are uncertain on account of the water-spouts or cloudbursts on the water-shed of the Gila River, which enters only 12 miles above Yuma. Frequently, within 24 hours, this river pours down as much water as the maximum flow of the Colorado. The periods of greater flow in the Colorado are between May 15th and August 15th; but the Gila is likely to rise at any time.

In repairing the break, practically all methods proven by the best previous practice were tried. Brush mattresses, about 18 in. in thickness, were put in, and more than 40 miles of cable, 2 000 cords of brush, and 1 100 piles were used, but neither mattresses nor anything else which could be placed upon that mud would hold, because the slightest current caused scour beneath the mattresses.

Finally, three parallel trestles, about 30 ft. apart, were built across the break, and from them were dumped the largest stones obtainable, some of which weighed as much as 6 tons. Such stones, lying upon the silt, had sufficient weight to prevent the water from undercutting, and held the silt until more stones could be put in.

Fortunately, several hundred cars, used in making the Lucian cut-off across Great Salt Lake, were at the disposal of the Southern Pacific Railroad. Most of these were 50-ton, side-dump, ballast cars, facetiously called "battleships." These were loaded with stone and brought down in trains of 40 or 50 cars. These cars were unloaded almost simultaneously on the trestles, and thus a cascade dam was made, with the large rocks below and the smaller ones on top. Of course, such a dam was permeable, but the rocks on the bottom held the silt until a tighter dam could be raised. Finally, when the flow was turned back into the old channel, the face of the dam was filled in with gravel, then with clay, and then, to make it water-tight, silt was pumped on it.

It is not thought that one trestle, or even two, would have been sufficient to make the closure, as the third was needed as a reinforcement. Some of the piles in the trestles were nearly 100 ft. long, many of them were 65 ft., and yet, during the last rise, the scouring power of the river was so great that many of these piles were undermined and shot up through the water and floated away. In the last closure of the break, water tanks were placed on some of the trestle bents and filled with water to keep them down and steady them.

The credit for handling this difficult work in such a masterly way should be given to Epes Randolph, Vice-President of the Southern Pacific Railroad, and his able assistant, H. T. Cory, M.

Mr. Washington.

Am. Soc. C. E., who was in charge of the work. The organization of men, teams, and material, and the railway transportation were almost perfect, and it should be remembered that the camp had to be created in a desert, and that more than half the laborers were Indians.

The greatest danger to be feared, now that the break has been closed, is that some part of the levee may give way.

In a letter received from Mr. Randolph about March 10th, 1907, the following details are given:

"The water was shut off by the application of 77 000 cu. yd. of material, mostly rock, but this did not by any means complete the dam.

"The total yardage dumped into the trestle up to March 1st is something like 130 000 cu. yd., clay and gravel being the principal material applied since the closure of the break. When the dam shall have been completed, it will contain a total of about 140 000 cu. yd., between 85 000 and 90 000 cu. yd. of which will be rock, but all the data will be given you accurately a little later."

The speaker has no doubt that had not this break been in the vicinity of a great railroad, and had not the entire resources of that railroad been at the disposal of the engineers, it would have remained open for years.

The New or Alamo River, in rushing across the country, has cut a channel about 80 miles long, some 1 500 ft. wide, and from 60 to 70 ft. deep. The Panama Canal is to be about 40 miles long, from 150 to 200 ft. wide on the bottom, and approximately 30 ft. deep; therefore the channel cut by the Colorado in a few months is approximately twice as long, nearly ten times as wide, and nearly twice as deep as the Panama Canal will be, and in that time the river has moved many times the yardage that will be required to complete the great interoceanic waterway.

The problem is almost as interesting from a financial standpoint, for the Statistician of the United States Reclamation Service has estimated that, had this break not been closed, the damage which would have been done eventually would have amounted to some $700 000 000.

Mr. Ockerson.

J. A. Ockerson, M. Am. Soc. C. E. (by letter).—Having made two visits to the scene of the "Colorado break," one just at the beginning of the closure work in August, 1906, and the other shortly after the failure in November of the same year, a few remarks relating thereto may not be amiss.

The natural ground on which the levee was built is composed of light loose soil, filled with cracks such as are found in silt deposited by overflow, which has afterward dried out. These cracks were filled, to a certain extent, with other light soil, probably

drifted in by the winds; to this add roots and fine drift extending down to water level, or even below, and it will be seen that the foundation for a levee was extremely bad. Mr. Ockerson.

After the break, a length of several hundred feet on both sides of the levee, and down to the water surface, was exposed. Several test-pits were also dug in the borrow-pit for the purpose of studying the cause of the failure. A careful examination of these held out little hope that, on such soil, a permanent levee could be constructed near the flowing river without the addition of sheet-piling below the water line.

The levee itself had little or nothing to do with the failure. The water stood but a short distance above its base at the time of the break; it found its way through the cracks and roots some distance below the base of the levee, and into the bed of the old intake in considerable volume while the levee was still intact. The defects developed over a distance of several hundred feet, and were of such a nature that no "sub-levee" or "hoop" would have availed.

Some attempts were made to stop the flow by dropping in, on the upper side, sacks of earth, brush, and bales of hay, but most of these passed through the "blow-outs" without accomplishing the desired purpose, and, in a short time, the levee itself fell into the chasms which had been washed out beneath it.

It is perhaps well to emphasize one phase of the situation having an important bearing on the failure. The increased slope to the Salton Sea had cut down the bed *via* the artificial intake, so that it was some 12 ft. below the bed of the Colorado River proper. Hence, when the water was diverted to the latter, it was high above the bed of the intake, and the intervening soil, of the character described, readily permitted a very considerable amount of percolation, and doubtless contributed largely to the disaster, as the distance between the two channels was not great.

It has been frequently asked why the old bed of the river, at the intake, is not also cut down by the effect of the current. The old bed was developed by the river itself, and is controlled by the functions of volume and slope, and, without changing these elements, the bed must remain essentially as it is.

That a serious mistake was made in taking the earth from the land side seems evident, but it is altogether probable that the break would have occurred if the borrow-pit had been on the river side of the levee, as the bed of the near intake channel was well below any borrow-pit.

It is said that a small muck ditch was dug near the toe of the levee on the river side, but it was neither wide enough nor deep enough to insure an impervious base, and it is doubtful if any muck ditch alone could have accomplished such a result. It is

Mr. Ockerson. possible that, in time, as now constructed, the subsoil will fill up with the finer sediment to such an extent as to put an effectual stop to serious leakage such as might result in another break; but the risk is too great, and the interests at stake are too important to rely on this expedient alone.

In comparing the work with that on other streams, it should be borne in mind that the soil has about the consistency of talcum powder when dry, and the stream has a slope down the intake of about 4 ft. to the mile. The problem of closure, therefore, is quite different from the conditions on the Atchafalaya or Missouri Rivers.

There can be little doubt, however, that several of the failures in the attempt to close the intake are traceable to defects in the methods used, and the failure to utilize ordinary engineering practice in such cases. Five of the failures were apparently due to neglecting the very important feature which is the key to success in work of that character, namely, that, first of all, the approaches to the work contemplated must be securely fixed. The banks and bed must be made stable beyond the possibility of erosion. Had this been done, the first effort to close the intake would have succeeded, and the cost would have been a few hundred dollars instead of the several hundred thousand which have been expended. The integrity of the "Rockwood gate" could probably have been preserved by the same means.

The two latest closures were certainly executed in as expeditious a manner as any previous work of such magnitude, and the engineers and others who participated in the work deserve great credit therefor.

Mr. Grunsky. C. E. GRUNSKY, M. AM. SOC. C. E. (by letter).—This paper was written in December, 1906, a few days after the levee forming a part of the first protection work had given way, and was transmitted to the Society under date of December 20th, 1906. This explanation seems to be necessary because it was not submitted at a Society meeting until March 20th, 1907, and at the time the advance copies reached the members, in February, 1907, the work of again turning the river had been accomplished. In order that this situation might be understood, a statement was read at the March meeting setting forth the essential additional facts, and giving credit to H. T. Cory, M. Am. Soc. C. E., the engineer in charge of the work, and to Colonel Epes Randolph, under whose direction Mr. Cory was operating, for the resourcefulness with which all obstacles were overcome, and for the successful accomplishment of a great work—the turning of a river, discharging at the time about 20 000 cu. ft. per sec., from a new course into its proper channel, under very unfavorable river conditions. The details of this work are full of interest and instruction for the

Mr. Grunsky.

profession, and a complete description thereof would be welcomed by the Society. To do this, however, is not within the province of the writer. It is hoped that a promised paper on this subject will be presented at an early date by Mr. Cory. In the meantime, but little is to be added to the original paper, the purpose of which was to explain the general situation and a possible great geographic change, and not to present or discuss the engineering problems connected with the closure of the Lower Mexican Heading. In the discussions which have been presented some reference is made to the amount of silt carried by the Colorado River. This amount is not known as accurately as would be desirable. The river is muddy, as its name implies, but the percentage of silt is hardly as great as Mr. Washington assumes. On this subject, Professor R. H. Forbes, of the Experiment Station at Tucson, Ariz., writes:

"On the basis of the profile constructed from available data for the volume of flow of the Colorado, and of the year's silt determination made in this laboratory, it is estimated, conservatively, that the river during 1900 brought down about 61 000 000 tons of sedimentary material."

This is only about 0.7% by weight of the river's flow for that year. According to figures published by the United States Reclamation Service in the Annual Report for 1902-03, the maximum silt contents of the river water at the spring freshet stage of 1903, was 1.97% by weight.

As additional information, which may now be submitted without encroaching upon that part of the subject which will be covered by Mr. Cory's paper, it is worthy of note that the dams closing the great break in the river bank showed no signs of weakness during the river's high stages in the spring of 1907, and that the river channel for many miles below the Lower Heading was so improved in its regimen by a long gradual rising stage that the river at its maximum stage for the season remained practically within banks and the levee was not wet throughout its full length; but, where tested by water standing against it, no underflow developed, except such slow percolation as must necessarily take place in the fine sandy soil of the country.

The levee, as finally carried out, is generally about 8 ft. high, has a top width of 10 ft., side slopes of 3 to 1, is covered with a layer of gravel, and carries a railroad track throughout its length. Under it there is a refilled "muck ditch" of varying depth.

The two dams closing the first and second breaks in the bank of the Colorado are known by the names of those who were in local charge of the work. The first is the "Hind" dam and the second the "Clarke" dam. The two structures are practically end to end, and appear as an enlarged section of the levee, which extends up and down stream from them.

Mr. Grunsky.

In reference to the elevation of the bed of Salton Sea, information has been received from Mr. M. Carvel Hall, of the United States Geological Survey, whose field work of 1905 made the production of a map of this region possible, that this elevation should be noted at 287 ft. instead of 278 ft., below sea level. On the same subject, W. B. Clapp, M. Am. Soc. C. E., also of the United States Geological Survey, has published a statement, that on October 4th, 1905, the New Liverpool Salt Company reported the greatest depth of water at 17 ft., and that the water surface was then at 256.5 ft. below sea level. Consequently, according to Mr. Clapp, the lowest point of the lake bed is 273.5 ft. below sea level. It will probably be near enough to call the elevation of the lake bed about 280 ft. below sea level.

The writer is in receipt of a letter from Dr. William P. Blake, now Professor of Geology, Emeritus, at the University of Tucson, Ariz., who states that he has heretofore suggested a name for the ancient lake of the Colorado desert. It is extremely appropriate that Dr. Blake, who was the first to describe this region, and who, at the early date of his examination, secured a clear understanding of the geographic and geologic features of this region, should give the ancient lake a name. "Cahuilla Lake," which he suggests, is appropriate, and should be generally accepted. It will not interfere with the use of the name, "Salton Sea," to designate the body of water now in the depression.

To supply an inadvertent omission from the original paper, it remains to be stated that the illustrations used were made available by the courtesy of the Reclamation Service. A few were from personal sketches, but all except two were from the official files.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 1052.

LIBERATION OF AIR IN SIPHONS.

By Charles Anthony, Jr., M. Am. Soc. C. E.

With Discussion by Messrs. P. M. Pritchard, and Charles Anthony, Jr.

Though siphons are appliances with which it is as well to have as little as possible to do, conditions may arise, especially when modifying pre-existing works, which compel their use. Under such circumstances, the designer is at considerable disadvantage through lack of data concerning the quantity of air which, under practical conditions, it is likely will be set free by water in its passage through a siphon.

Last year the writer designed and erected at East London, Cape Colony, the 12-in. siphon shown in Fig. 1.

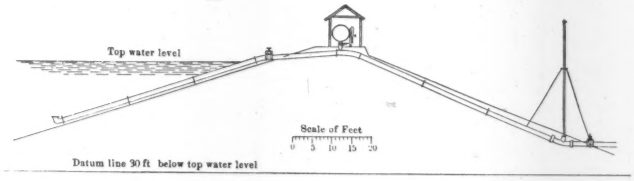


Fig. 1.

Before leaving Cape Colony he had not time to carry out, as he had intended, a series of experiments with the object of ascertaining the quantity of air liberated under different conditions, as

regards the amount of negative head, temperature of water, amount of dissolved gases, and quantity of water passed, with the object of framing a formula which would take into due account each of these factors; yet the following results of the only experiment effected may, in view of the entire lack of data on the subject, be of some little service.

12-in. Siphon Over the Amalinda Embankment.

Air vessel charged February 12th, 1906, level of water inside	=	419.50
Air vessel empty March 22d, 1906, level of water inside	=	415.50
Capacity of air vessel	=	650 gal.
Quantity of water supplied during above period of 39 days, $39 \times 359\,000$, approximately	=	14 000 000 gal.
Level of water in reservoir, February 12th, 1906	=	404.50
Level of water in reservoir, March 22d, 1906	=	409.50
Mean level of water in reservoir, during period of 39 days	=	407.00
Mean level of water in air vessel during period of 39 days	=	417.50
Quantity of air liberated within air vessel during period of 39 days, reduced to atmospheric pressure, $\frac{650\,[34 - (415.50 - 409.50)]}{34}$	=	535 gal.
Air liberated, in percentage of water supplied, $\frac{100\% \times 535}{14\,000\,000}$	=	0.0038%
Under a mean vacuum of 417.50 — 407.00	—	10.5 ft.

This experiment indicates that water containing the quantity of dissolved gases ordinarily found in impounding reservoirs in South Africa, in latitude 33°, under a mean temperature of about 70° fahr., that is, some 4% by volume, when passed at the rate of about 360 000 gal. per day, through a 12-in. siphon, under a negative head of 10 ft. 6 in., liberates, approximately, 0.00004 of its bulk of air, at atmospheric pressure.

DISCUSSION.

P. M. PRITCHARD, ASSOC. M. AM. SOC. C. E. (by letter).—It would appear to the writer to be a somewhat difficult matter to estimate, even approximately, the quantity of air liberated from water flowing through a siphon, owing to the fact that some of the air, freed from solution in the water by the reduction of pressure, remains entrained in the water and is carried with it through the siphon. The quantity of air or gas held in solution by the water and liberated for a given reduction of pressure can be found by a simple laboratory experiment. Mr. Pritchard.

The writer has recently installed a siphon of considerable size, as shown by Fig. 2. In this case, a receiver was fixed near the

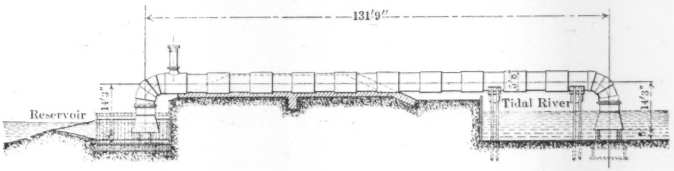


FIG. 2.

delivery end of the siphon with the object of catching air liberated from the water and swept along the top of the siphon pipe.

At present a small steam jet extracts the air collected in the receiver while the siphon is at work, but, from the appearance of the water delivered by the siphon, it is evident that a large quantity of the air liberated from the water is entrained and carried away with it.

It seems that it ought to be possible to design an ejector, operated by the water itself, to exhaust the air accumulated in the receiver, and experiments are being made on these lines.

Incidentally, the writer would say that the siphon shown in Fig. 2 is the largest of which he has been able to find any record. With a 6-in. difference of water level, the delivery, very approximately, is 1 500 000 English gal. per hour.

CHARLES ANTHONY, JR., M. AM. SOC. C. E. (by letter).—The dearth of discussion, or further data on the subject, is the best apology for the writer's brief note on the liberation of air in siphons. He regrets that he is unable to furnish additional information, but may observe that Mr. P. M. Pritchard has evidently missed the point at issue, when he states that: Mr. Anthony.

"The quantity of air or gas held in solution by the water and liberated for a given reduction of pressure can be found by a sim-

Mr. Anthony.

ple laboratory experiment," while it is "a somewhat difficult matter to estimate, even approximately, the quantity of air liberated from water flowing through a siphon, owing to the fact that some of the air, freed from solution * * * by the reduction of pressure, remains entrained in the water and is carried with it through the siphon."

Although these are truisms, their presentment argues a misapprehension, on the part of Mr. Pritchard, of the data required when designing air vessels or other apparatus for clearing siphons of air, the amount liberated and retained in the siphons themselves under actual working condition being the point on which further data are much needed.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 1053.

THE LAWS OF PROPORTIONING CONCRETE.*

By William B. Fuller and Sanford E. Thompson,
Members, Am. Soc. C. E.

With Discussion by Messrs. James L. Davis, H. de B. Parsons, Allen Hazen, Samuel Tobias Wagner, R. Feret, W. F. Dennis, Richard H. Gaines, William B. Fuller and Sanford E. Thompson.

Experiments made by one of the authors at Little Falls, N. J., in 1901, upon the density and transverse strength of concrete beams mixed in various proportions, by weight, ranging in proportions from 1 : 0 to 1 : 6 : 10, indicated that the strength of concrete varies with the percentage of cement contained in a unit volume of the set concrete, and also with the density of the specimen. With the same percentage of cement in a given volume of concrete, the densest mixture, irrespective of the relative proportions of the sand and stone, was in general the strongest.

The Little Falls tests further indicated that for the materials used there was a certain mixture of sizes of grains of the aggregate which, with a given percentage, by weight, of cement, to the total aggregate, gave the highest breaking strength. In practice, also, it was found that the concrete made with this mixture worked most smoothly in placing. The mixture of sizes of particles of aggregate which appeared to give the best results gave for its mechanical analysis a curve approaching a parabola, with its begin-

* Presented at the meeting of April 17th, 1907.

ning at zero of co-ordinates, and passing through the intersection of the curve of the coarsest stone with the 100% line, that is, passing through the upper end of the coarsest stone curve.

Graded stone of the same maximum diameter and character was used in all the Little Falls experiments, and the laws mentioned were discovered by a comparison of the final results of the tests rather than by logical investigation.

These results were brought to the attention of the Aqueduct Commission of the City of New York in 1903 by J. Waldo Smith, M. Am. Soc. C. E., then Chief Engineer for the Commission, and at his suggestion the Commission authorized a further study of these and related matters, the results of which study are here presented.

The experiments to be mentioned were carried on at Jerome Park Reservoir, in the Bronx District, where a large amount of concrete construction was being done by contract under the supervision of the Commission, and cement, sand and stone were readily available. A large, light and warm room was provided as a laboratory, and the necessary apparatus was purchased. Sufficient labor and other assistance was always obtainable from the Contractors or from the Engineering and Inspection staff on the Reservoir Construction.

As has been stated, previous tests indicated that the greatest strength from any given percentage of cement was obtained when the concrete was of the greatest possible density, that is, having the least percentage of voids, and further, that the greatest density was obtained when all the materials were proportioned so as to give a regular mechanical analysis curve approaching a parabola. It also appeared probable that concrete of the greatest density would be the least permeable to water.

With the foregoing laws established, it would be possible to compare the values of different aggregates and various proportions of the same aggregates by volumetric tests, the best mixture in general giving the smallest volume of concrete.

The tests were accordingly arranged with the view of fixing the limitations of this theory, to ascertain the effect of different aggregates upon the density, strength and permeability of concrete, and particularly to determine the exact sizes of aggregate which,

mixed together with a given proportion of cement, would form the best concrete. The density and strength of concrete made up of an aggregate of ideal mechanical analysis and of different maximum size was compared with the density and strength of concrete made of average sand and stone or gravel in ordinary nominal mixtures.

The relative permeability of concrete with different aggregates and different proportions of cement was also studied.

It also became necessary to study many related subjects, such as the physical characteristics of different cements, broken stone, broken-stone screenings, gravel and sand, especially with reference to specific gravity, weight, voids, density, etc. Each material was tested by itself and when mixed in various proportions with others.

The experiments were begun with a series of tests of the density of different mixtures of aggregates and cement, to study the laws of proportioning for maximum density with different materials, and these density experiments were followed by the manufacture of concrete beams for comparing the laws of strength with the laws of density, and determining the relation between these two causes. As the compressive strength of concrete is a truer measure of its quality than the transverse strength—which in the case of concrete is really one form of tension test—two pieces of each beam after being broken in the beam machine were capped with neat cement so as to form prisms, and these were tested for compressive strength; a number of them were also tested for the compressive modulus of elasticity. A selected number of pieces of the broken beams were also tested for permeability.

Other secondary experiments, upon the density of mortars and the quantity of water required for different sizes of sand, were begun, and although not extended far enough to reach definite conclusions, the results are of interest as indicating the possibilities of another important line of investigation. It was intended to use these results further as preliminary to a series of permeability tests of mortars composed of cement and sand having grains of different size, and containing admixtures of puzzolan cement and of hydrated lime.

Conclusions.

The following conclusions have been drawn from the tests. Many of them introduce laws which the writers believe have not

hitherto been recognized, and are opposed in certain cases to current ideas. Although based upon experiments with but two aggregates, these were so characteristic as to make the various comparative results of nearly general application.

Unless otherwise designated, the percentage of cement and the relative grading of the aggregates remain the same.

1.—Stone of the largest size makes the strongest concrete under both compression and transverse loading, *i. e.*, a graded aggregate in which the maximum size of the stone is 2½ in. in diameter gives stronger concrete than a graded aggregate with 1-in. maximum size, and the 1-in. stone gives a stronger concrete than ½-in. stone. A concrete in which the graded aggregate runs to 1 in. in maximum size will require for equal strength about one-sixth more cement, and with an aggregate running to ½ in. maximum size about one-third more cement than concrete with an aggregate in which the maximum size is 2½ in.

2.—The largest stone makes the densest concrete. Concrete made with graded stone having a maximum diameter of 2½ in. is noticeably denser than that with 1-in. stone, and this is denser than that with ½-in. stone.

3.—Round material like gravel, under similar conditions, gives a denser concrete than broken stone.

4.—Sand produces a denser concrete than screenings of similar sized grains.

5.—A concrete with an angular coarse aggregate, such as broken stone, is stronger than one with a rounded coarse aggregate, like gravel, and the same sand and cement—although the rounded aggregate produces greater density—thus indicating a stronger adhesion of cement to broken stone than to gravel. However, if the sand is also angular, like screenings, but with its grains of the same sizes as the sand, the concrete with rounded coarse and fine aggregate is the stronger, probably because of its greater density.

6.—Aggregates in which particles have been specially graded in sizes so as to give, when water and cement are added, an artificial mixture of greatest density, produce concrete of higher strength than mixtures of cement and natural materials in similar proportions. The average improvement in strength by artificial

grading under the conditions of the tests was about 14 per cent. Comparing the tests of strength of concrete having different percentages of cement, it is found that for similar strength the best artificially graded aggregate would require about 12% less cement than like mixtures of natural materials.

7.—The strength and density of concrete is affected but slightly, if at all, by decreasing the quantity of the medium size stone of the aggregate and increasing the quantity of the coarsest stone. An excess of stone of medium size, on the other hand, appreciably decreases the density and strength of the concrete.

8.—The strength and density of concrete is affected by the variation in the diameter of the particles of sand more than by variation in the diameters of the stone particles.

9.—An excess of fine or of medium sand decreases the density and also the strength of the concrete, as will also a deficiency of fine grains of sand in a lean concrete.

10.—The substitution of cement for fine sand does not affect the density of the mixture, but increases the strength, although in a slightly smaller ratio than the increase in the ratio of cement.

11.—It follows from the foregoing conclusions that the correct proportioning of concrete for strength consists in finding, with any percentage of cement, a concrete mixture of maximum density, and increasing or decreasing the cement by substituting it for the fine particles in the sand or *vice versa*.*

12.—In ordinary proportioning with a given sand and stone and a given percentage of cement, the densest and strongest mixture is attained when the volume of the mixture of sand, cement and water is so small as just to fill the voids in the stone. In other words, in practical construction, use as small a proportion of sand and as large a proportion of stone as is possible without producing visible voids in the concrete.

13.—The best mixture of cement and aggregate has a mechanical analysis curve† resembling a parabola, which is a combination of a curve approaching an ellipse for the sand portion and a tangent straight line for the stone portion. The ellipse runs to a diameter of one-tenth of the diameter of the maximum size of stone, and the stone from this point is uniformly graded.

* This very important law requires further tests for confirmation, outside of the limits of the present tests.

† For definition of mechanical analysis, see page 73.

14.—The ideal mechanical analysis curve, *i. e.*, the best curve, is slightly different for different materials. Cowe Bay sand and gravel, for example, pack closer than Jerome Park stone and screenings, and therefore require less of the size of grain which the writers designate as sand.

15.—The form of the best analysis curve for any given material is nearly the same for all sizes of stone, that is, the curve for ½-in., 1-in., and 2½-in. maximum stone may be described by an equation with the maximum diameter as the only variable. In other words, suppose a diagram in which the left ordinate is zero, and the extreme right ordinate corresponds to 2½-in. stone, with the best curve for this stone drawn upon it. If, now, on this diagram the vertical scale remains the same, but the horizontal scale is increased two and a quarter times, so that the diameter of 1-in. stone corresponds to the extreme right-hand ordinate, the best curve for the 1-in. stone will be very nearly the one already drawn for the 2½-in. stone. The chief difference between the two is that the larger size of stone requires a slightly higher curve in the fine sand portion.

16.—It follows from this last conclusion that, from a scientific standpoint, the term "sand" is a relative one. With 2½-in. stone, the best sand would range in size from 0 to 0.22 in. in diameter, while the best sand for ½-in. stone would range in size from 0 to 0.05 in. in diameter.

17.—The permeability or flow of water through concrete is less as the percentage of cement is increased, and in very much larger inverse ratio.

18.—The permeability is less as the maximum size of the stone is greater. Concrete with maximum size stone of 2½-in. diameter is, in general, less permeable than that with 1-in. maximum diameter stone, and this is less permeable than that with ½-in. stone.

19.—Concrete of cement, sand and gravel is less permeable than concrete of cement, screenings and broken stone; that is, for equal permeability, a slightly smaller quantity of cement is required with rounded aggregates like gravel than with sharp aggregates like broken stone.

20.—Concrete of mixed broken stone, sand and cement is more permeable than concrete of gravel, sand and cement, and less per-

meable than similar concrete of broken stone, screenings and cement; that is, for water-tightness, less cement is required with rounded sand and gravel than with broken stone and screenings.

21.—The permeability decreases materially with age; increases nearly uniformly with the increase in pressure; and increases as the thickness of the concrete decreases, but in a much larger inverse ratio.

Mechanical Analysis.*

Mechanical analysis consists in separating the particles or grains of a sample of any material—such as broken stone, gravel, sand or cement—into the various sizes of which it is composed, so that the material may be represented by a curve, each ordinate of which is the percentage of the weight of the total sample which passes a sieve having holes of a diameter represented by the distance of this ordinate from the origin in the diagram.

The objects of mechanical analysis curves, as applied to concrete aggregates, are:

(1) To show graphically the sizes and relative sizes of the particles;
(2) To indicate what sized particles are needed to make the aggregate more nearly perfect and so enable the engineer to improve it by the addition or substitution of another material; and
(3) To afford means for determining the best proportions of different aggregates.

Materials.

Cement.—Portland cement from the regular shipments to the reservoir was used in all except a few comparative tests of different brands. This cement had been tested in the regular fashion for reservoir work, and, in addition, every bag which was used in the later experiments was subjected to the purity test. It was found that the cement used in the 1904 beams, which did not set up satisfactorily, failed to pass this test, while all the cement passing this test appeared to be satisfactory from a chemical standpoint.

* A description of the methods and apparatus for mechanical analysis, and for plotting the curves, may be found in the chapter by Mr. Fuller on proportioning concrete, in Taylor and Thompson's "Concrete, Plain and Reinforced."

Purity Test.—The purity test is as follows:

Provide a glass-stoppered bottle of muriatic acid, two shallow white bowls or two ½-in. by 6-in. test tubes, a glass rod, and a pair of rubber gloves. Put in a bowl or a tube as much cement as can be taken on a nickel 5-cent piece; moisten it with half a teaspoonful of water; cover with clear muriatic acid poured slowly upon the cement while stirring it with the glass rod. Pure Portland cement will effervesce slightly, give off some pungent gas, and gradually form a bright yellow jelly without any sediment. Powdered limestone or powdered cement-rock mixed with the pure cement will cause a violent effervescence, the acid boiling and giving off strong fumes until all the carbonate of lime has been consumed, when the bright yellow jelly will form. Powdered sand or quartz or silica mixed with cement will produce no other effect than to remain undissolved as a sediment at the bottom of the yellow jelly. Reject cement which has either of these adulterants.*

The results of tension tests on a sample of the cement used were as follows:

	Age, in days.		Age, in months.					
	1	7	1	2	3	4	5	6
Average tensile strength, in pounds per square inch	218	556	658	686	708	656	738	754

Aggregate.—Two classes of materials were used for the aggregate: broken stone and screenings from the crushers at the reservoir, and Cowe Bay gravel and sand dredged from the river.

The stone as it came from the crushers at Jerome Park was run through revolving screens to remove stones of greater diameter than 2 in., which were not permitted in the reservoir lining, and to separate the screenings, which were measured separately from the broken stone in proportioning the concrete.

The 1904 beams were made with material as it came from the crusher screens, this series being for approximate comparison of the strength of different proportions, while the density experiments

* Judson's "City Roads and Pavements," 1902.

were in progress. The density tests were a necessary preliminary to the more scientific mixtures made in 1905.

For the density tests and the specimens made in 1905, the stone and screenings were separated in the laboratory by twenty-one sieves ranging in size from 3-in. openings to No. 200 mesh—the latter corresponding to an opening of 0.0027 in. By using different mixtures of the sizes separated by these sieves, it was possible to obtain an infinite variety of proportions.

Character of Jerome Park Rock.—The rock at Jerome Park is technically a mica schist, containing about 35% mica, although the mica is in such a state that in concrete or mortar it does not form planes which affect the strength seriously. The variation in the quality of the rock in different parts of the reservoir gave some trouble in the laboratory, and led to less accuracy in the results of the density experiments, and in forming the proportions of the mixtures, than if the specific gravity had been uniform. The most noticeable difference in different ledges was in the predominance of quartz, some of the lots being lighter in color than the others. Since the specific gravity of the minerals in the native rock ranged from 2.6 to 3.2, the specific gravity of different lots of screenings varied appreciably, and the specific gravity of different diameters of the same rock also varied because some of the minerals of which the rock was composed crushed more readily than others, and therefore certain sizes contained predominating minerals which determined their specific gravities.

Character of Cowe Bay Materials.—The material from Cowe Bay is a water-worn rounded bank gravel and sand, thoroughly clean, and consisting almost entirely of quartz particles. Its specific gravity is quite uniform, being about 2.65.

Mechanical Analyses of Aggregates.—The average mechanical analyses of these materials are given in Table 1.

Screening Aggregate in Laboratory.—Large sieves, about 2 ft. square, were used for screening the aggregate into the twenty-one sizes. The sieves rested in a frame, one above another, seven to a frame, so that any one of them could be pulled out like a drawer without disturbing the others. The frames rested upon a pair of rockers consisting of 2-in. plank, sawed with angles instead of rounding, so that the motion of the sieves when rocked by hand

was a combined slide and jar. Spikes were driven into the rockers at the angles to make them more durable. A frame of sieves and a single sieve are shown in the background of the photograph, Fig. 2, Plate X, and the details are shown in Fig. 1.

TABLE 1.

Diameter of opening in sieves, in inches.	Jerome Park Material.				Cowe Bay Sand and Stone.			
	2½-in. stone, percentage passing.	1-in. stone, percentage passing.	½-in. stone, percentage passing.	Screenings, percentage passing.	2½-in. stone, percentage passing.	1-in. stone, percentage passing.	½ in. stone, percentage passing.	Sand, percentage passing.
2.25	100.0	.....	.....	.....	100.0	.....	.....	100.0
1.50	78.4	.....	.....	.....	94.2	.....	.....	100.0
1.00	41.8	100.0	.....	.....	81.8	100.00	.....	100.0
0.75	29.9	71.5	.....	.....	67.3	82.27	.....	100.0
0.60	18.4	44.2	.....	100.0	44.8	54.77	.....	97.9
0.48	11.4	27.3	100.0	99.7	26.4	32.28	100.00	96.2
0.36	6.8	16.3	59.6	99.1	14.0	17.12	53.03	95.0
0.29	3.6	8.6	31.6	96.9	8.6	10.51	32.57	93.7
0.20	1.9	4.5	16.7	92.8	5.05	6.17	19.13	91.8
0.16	1.3	3.1	11.4	87.2	3.40	4.16	12.88	88.9
0.10	1.0	2.4	8.7	80.7	1.76	2.15	6.67	85.1
0.075	0.67	1.6	5.8	76.7	1.57	1.92	5.95	78.9
0.046	0.61	1.4	5.3	70.8	1.38	1.69	5.23	68.3
0.034	0.58	1.39	5.0	66.3	1.28	1.56	4.85	62.4
0.020	0.55	1.32	4.8	55.8	1.09	1.33	4.13	49.9
0.016	0.48	1.15	4.2	47.8	0.98	1.20	3.71	37.3
0.014	0.47	1.12	4.1	43.2	0.80	0.98	3.03	31.8
0.0071	0.32	0.76	2.8	21.0	0.47	0.57	1.78	7.1
0.0058	0.27	0.64	2.3	15.4	0.43	0.53	1.63	4.6
0.0036	0.19	0.45	1.6	6.7	0.26	0.32	0.98	2.3
0.0027	0.01	0.24	0.8	2.3	0.01	0.01	0.04	1.3

This apparatus worked fairly well, although difficulty was experienced in getting uniformity of screenings. It is suggested that for future work some form of pedometer be placed upon the frames to record the number of shakes, which number should be definitely fixed for each series of sizes. Each lot of unscreened material placed in the top sieve of each series should be measured so as not to overload the apparatus.

For testing small samples of aggregate, and for cement, the apparatus shown in Fig. 1, Plate X, was used.

Diameters of Aggregates.—The diameters of particles in the tables and diagrams in this paper are designated by the size of the openings in the sieve, by the diameter of the largest particles passing through it, instead of by the commercial number of the sieve.

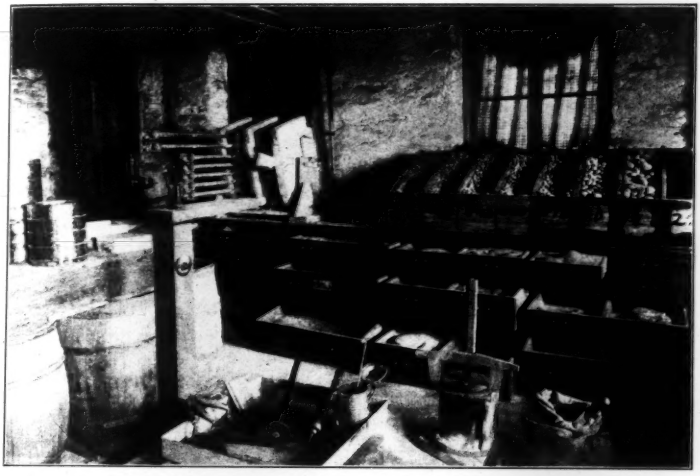


FIG. 1.—APPARATUS USED IN VOLUMETRIC TESTS.

FIG. 2.—IMPLEMENTS USED IN MAKING BEAMS.

The diameters of particles, with the corresponding commercial sieve numbers used in this series of tests, are shown in Table 2.

TABLE 2.—SIZES OF SIEVES.

Normal sizes of sieves, in inches.	Diameters passing sieves, in inches.	Normal sizes of sieves, in inches.	Diameters passing sieves, in inches.	Commercial numbers of sieves, in inches.	Diameters passing sieves, in inches.	Commercial numbers of sieves, in inches.	Diameters passing sieves, in inches.
2.25	2.25	0.35	0.36	10	0.075	50	0.014
1.50	1.50	0.27	0.29	15	0.046	74	0.0071
1.00	1.00	0.20	0.20	20	0.034	100	0.0058
0.75	0.75	0.15	0.16	30	0.020	150	0.0036
0.60	0.60	0.10	0.10	40	0.016	200	0.0027
0.45	0.48						

The number of sieves adopted was so large that it required considerable time and labor in making each test. In future experiments, it may be possible to omit a few of the sizes, if tests of the materials indicate that this can be done without appreciable effect upon the result.

VOLUMETRIC TESTS.

Methods and Apparatus for Determining Density.—As a preliminary to the beam experiments, nearly 400 trial mixes of aggregates, composed of various sized particles, following a logical plan, and containing first 10% of cement to the total weight of the dry materials and then other percentages of cement, were made and mixed with water to the same medium wet consistency, and the resulting volumes thus obtained from exactly the same total weight of dry materials (corrected for specific gravity) were compared. The tests of density are termed volumetric tests in this paper. The general procedure (described below) in making them is that adopted by the French Commission, in 1894, and the volumes of material were calculated by methods used by M. R. Feret, of Boulogne-sur-Mer, France, in his determination of elementary and of absolute volumes. The apparatus used in these tests is shown in Fig. 1, Plate X.

Weighing.—All materials were proportioned by dry weight. The stone and screenings were brought to the laboratory, and the screen-

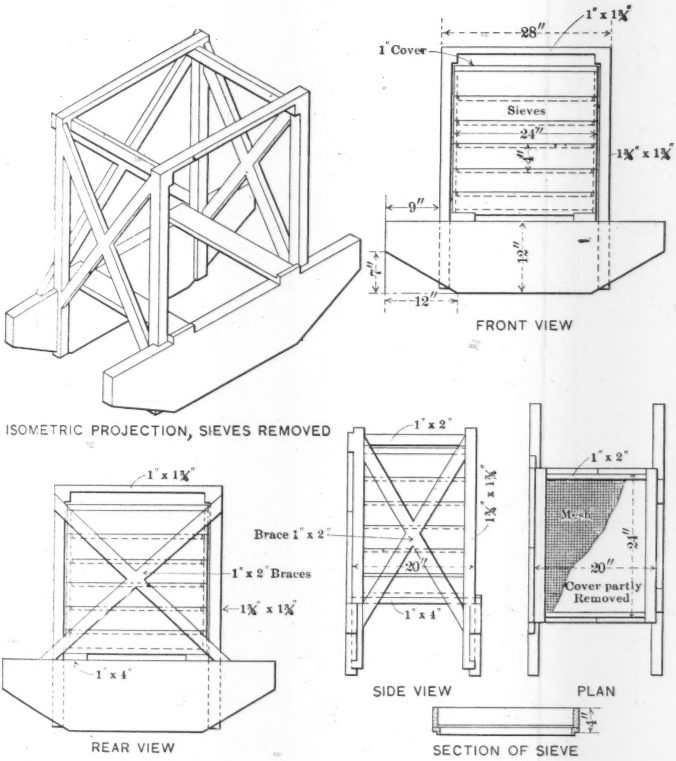
ings dried in large pans on the stove. To hasten the drying, they were stirred continually, and, when dry fine dust freely arose from them, tests indicated that they contained no appreciable moisture. Tests showed that material stored in the laboratory for some time during the winter did not collect enough moisture to affect the weights used in the experiments.

For weighing the materials, a Fairbanks scale (No. 1 288) with a compound beam having seven scales, was used. The scales read to half pounds, and, with a wire rider made in the laboratory, it was possible to read to hundredths of pounds, although the accuracy was not much greater than tenths. In the density tests, the materials were weighed directly in the mixing pan placed upon the platform of the scales. The water was usually weighed in a 16-qt. galvanized-iron pail.

Measuring.—For measuring the volume of concrete made in the volumetric tests, an old cast-iron air-brake cylinder and piston was found convenient. The cylinder was 8-in. in diameter, inside measurement, flanged at both ends, with a blank flange bolted to it, thus forming a vessel 8 in. in diameter and 9 in. deep. This vessel was calibrated carefully, and its contents calculated accurately for each unit of depth. After placing the concrete in the cylinder, the piston plunger was pressed down upon it until its bottom surface coincided exactly with the surface of the concrete. The depth of the concrete was then determined, and from this depth its volume was readily calculated.

A larger cylinder, 12 in. in diameter and 18 in. deep, was made for volumetric tests of materials containing large sized stone, but comparative tests with this and the smaller cylinder showed that the densities obtained by the two apparatus were so nearly identical that the larger one was not used to any great extent.

The materials were proportioned by mechanical analysis curves, as described in preceding paragraphs, and the weights of each diameter were scheduled from these curves. The mixing pan, which was about 2 ft. square, and 3 in. deep, was placed upon the scales, and any trowels and rammers or other apparatus to be used in the operation, and therefore to be more or less coated with cement, were placed in the pan, and the weight of this tare recorded on one of the beams. The aggregates, beginning with those of the finest diame-



DETAIL OF FRAME FOR SIEVES FOR SCREENING AGGREGATES IN LABORATORY.

FIG. 1.

ter, were then weighed. The schedule of weights of the aggregate was made up so that each weight included the weight of all the finer aggregates. In this way the weighing poise was reset for each size of material, and the materials were placed in the pan, one on top of the other. The cement was weighed last, so as to be on top of the other materials.

Mixing.—The mixing was done by two men working on opposite sides with large trowels, and turning the dry material until it was of uniform color, in the same manner as concrete is turned by hand. The material was then formed in a ring, water was poured into the center, and the mass was turned until the mixing was thorough. The mixing pan, trowels, shovel (in plan and elevation), cleaner (in plan and section), and rammer are sketched in Fig. 2.

Because of the variation in the sizes of the grains of the aggregate in the different volumetric mixes, it was impossible to select a definite percentage of water to be used in all the tests, or to select in advance definite percentages for each mix. The mixtures containing the largest quantities of very fine material required the largest percentages of water.

The water, therefore, was added by judgment to obtain a soft mushy mixture which would scarcely hold its form in the mixing pan, but which was not so fluid that the mortar would run away from the stones. A pail of water, with its dipper, was first weighed, as much water as required was added, and the weight of the remaining quantity was deducted from the original weight to determine the net weight used. As the surplus water was removed from the specimen after placing it in the cylinder, the quantity finally given in the density sheets represents the water actually contained in the specimen as it began to set.

Ramming.—The mixed concrete which had been previously weighed, was introduced into the cylinder and rammed in 2-in. layers. This frequent ramming was necessary because of the small size of the receptacle, the friction on the sides preventing the material from settling even with very wet mixtures if ramming was delayed until the entire amount, which averaged a little over 6 in. in thickness, was placed. The rammer which was found best was a cast-iron disk, about 4 in. in diameter, with an upright handle, shown in Fig. 2.

A mixture of the consistency described above gave the best results in the cylinder. If too dry, there was always a possibility of occasional large air voids, and it was more difficult to manipulate, although the resulting rammed volume was about the same as the mushy mix which was used. A very wet mixture did not work

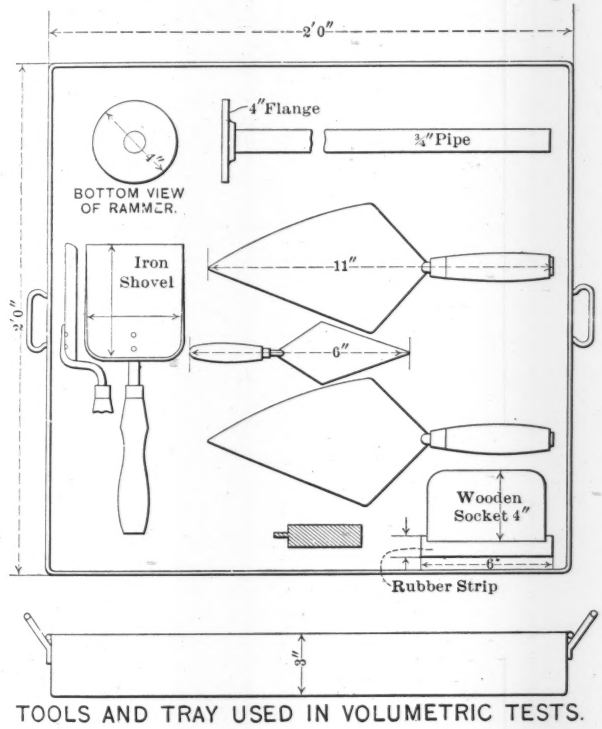


FIG. 2.

quite as well as either the mushy or the dry mixtures, the mortar probably not being in a sufficiently plastic condition to lubricate the stones. Rather curiously, a very wet mixture required more fine material to fill the voids than the others.

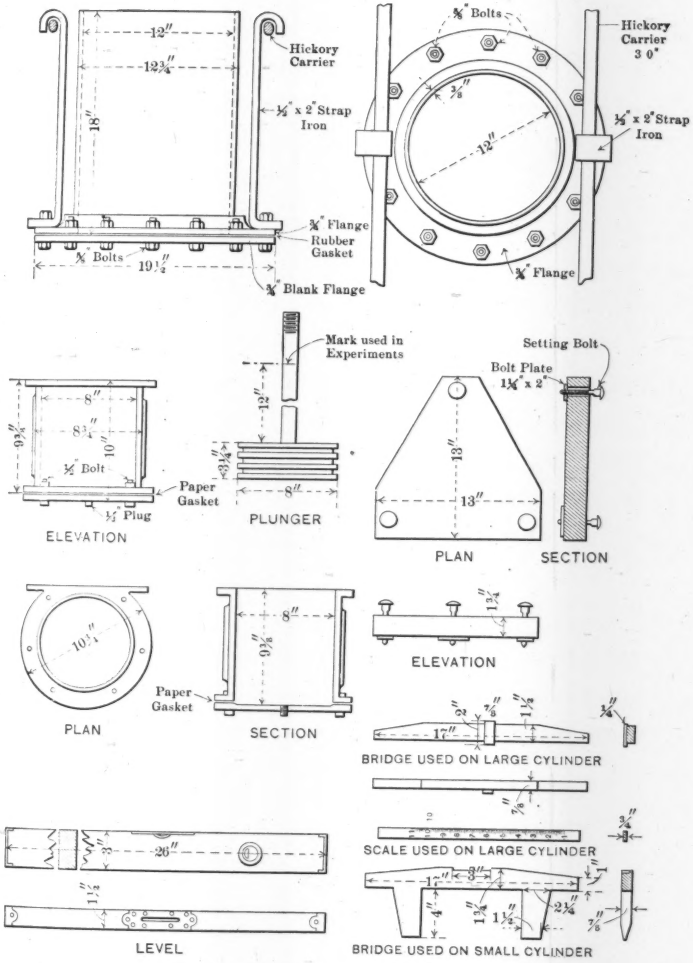
Removing Surplus Water.—As the concrete was rammed, the surplus mortar, if there was any, rose to the surface, and the water

separated, forming a layer from $\frac{1}{8}$ to $\frac{1}{4}$ in. in depth. After this had become clear, it was removed by a small suction pump, the weight being deducted from the weight of the water used in the mix.

Final Weighing.—The cylinder containing the concrete was weighed, as a check upon the weights of the dry material and the water; the mixing tray, together with the tools, was weighed, and the quantity of material adhering to them was thus found, being the difference between this weight and the weight of the clean tray and tools. The weight of the portion of the mix adhering was introduced into the final calculations, as described below.

Measurement.—After the surplus water was removed, care was taken to see that the surface of the concrete was level, with no projecting stones, and the cylinder was leveled by placing it upon a tripod consisting of a board with three lag-screws for legs. The piston was introduced and pressed firmly down, but not with sufficient force to disturb the mortar on top and thus force it up between the piston and the side of the cylinder. The wooden yoke was placed in position, and the depth of the concrete in the cylinder was measured by taking the height from the top of the yoke to a mark on the piston. The cylinder, piston, yoke, scale and magnifying glass are shown in Fig. 3. The test was now complete and ready for computation, the records having been entered upon the blank form, Table 3. The concrete was thrown away, and the tools were cleaned, ready for the next experiment.

Recording and Computing Data.—The form for recording the data in the volumetric tests of concrete—Table 3—is given with a typical test of Cowe Bay material recorded upon it, the observed and calculated figures being in italics. It is arranged so that all the weights may be entered in the laboratory, and the column of items also gives the constants to be used in calculation and the method of combining the various items so that the calculation of each sheet may be made by rote by an unskilled computer. The experiments in each class were numbered consecutively, for convenience in reference. The items recorded in the laboratory were the weights, and the depths of the piston in the cylinder—Items 1 to 17, 24, and 34 to 40. The figures following many of the items refer to the numbers of other items; the fraction following Item 19 represents, for example, the portion of the mix adhering to the tray and tools



APPARATUS USED IN VOLUMETRIC TESTS.

FIG. 8.

TABLE 3.—FORM FOR RECORDING VOLUMETRIC TESTS.

1. No.	377½
2. Date	3/10/05
3. Nominal Mix.	
4. Kind of Cement	Giant No. 20.
5. Weight	2.65
6. Wt. of Aggreg. finer than 0.0071"	0.85
7. " " " coarser " "	21.50
8. " " Vessel + Water (1)	29.00
9. " " " " (2)	27.40
10. " " Water Used	1.60
11. Total Wt. Mixed	26.00
12. Wt. of Tray + Tools (2)	13.15
13. " " " " (1)	13.08
14. " " Mixed Adhering	0.07
15. " Syringe + Water	2.62
16. " Syringe	2.60
17. " Free Water	0.02
18. Net Mix Set = 11 − 14 − 17	26.51
19. Water left on Tray $\frac{10 \times 14}{5+6+10}$	0.02
20. Net Water Set = 10 − 17 − 19	1.58
21. Net Cement " = $5 - \frac{5 \times 14}{5+6+10}$	2.61
22. " Aggreg. finer than 0.0071 = $6 \times \left\{ \frac{6 \times 14}{5+6+10} \right\}$	0.84
23. Net Aggreg. coarser than 0.0071	21.50
24. Depth of concrete in Cylinder, in	5.99
25. Vol. of concrete in Cylinder, in. 0.0293 × 24 − 0.001	0.174
26. Net Water per cu. ft. as mixed $\frac{10}{25}$	9.2
27. " " " " " " set $\frac{20}{25}$	9.1
28. " Cement " " " " " $\frac{21}{25}$	15.0
29. " Aggreg. " " " " " $\frac{22+23}{25}$	128.4
30. Abs. Vol. Water per cu. ft. as set $\frac{27}{62.4}$	0.146
31. " " Cement " " " " " $\frac{28}{194}$	0.077
32. " " Aggreg. " " " " " $\frac{29}{165}$	0.778
33. " " Total 30 + 31 + 32	1.001
34. Wt. of Form + Concrete	77.23
35. " " Form	50.75
36. " " Concrete	26.48
37.	
38. Temp. of Water	40°
39. Time of Mixing after Wetting	
40. Remarks on Consistency	

REMARKS.—*Looked little stony in mix. Top surface filled. No excess of mortar on top. Small stones. No. 10 right under top surface.*

which is water. The assumption is made—the fact having been determined by experiment—that the mortar sticking to the tray and tools consists of cement and particles of aggregate finer than diameter 0.0071, and water. The weight of the water in this mortar which adheres may be found from the proportion:

Mix adhering: total fine mortar = water in mix adhering: total water. Expressed in item numbers, this becomes:

$$\text{Item } 19 = \frac{\text{Item } 14}{\text{Item } 5 + \text{Item } 6 + \text{Item } 10} \times \text{Item } 10.$$

The net water contained in the concrete is thus Item 20, which equals Item 10—(Item 17 + Item 19).

The absolute volumes, Items 30 to 32, which represent the total volume of the liquid, or the volumes of the grains of cement or aggregate, are the net volumes per cubic foot divided by the specific gravity of each of the materials. These absolute volumes represent simply the ratios of the actual volumes of each ingredient to the total volume of the concrete, and are the figures by which the different mixes are compared.

Proportioning the Ingredients for Maximum Density.

The volumetric tests of concrete were begun in 1904. In the first place, it was the aim to determine, for the various materials under consideration, artificial mixtures of aggregates, of diameters graded by methods of mechanical analysis already described, which would give concrete of maximum density. These ideal mixes having been determined, the concrete made from them could be compared with respect to density, strength and permeability, with concrete made from simple mixtures of natural materials, and thus the best proportions to use with different materials upon actual construction work could be fixed.

Most of the experiments were made upon the same proportions of total aggregate to cement. The tests with Jerome Park stone and screenings for the most part were with 10%, by weight, of cement to the total dry materials. The tests with Cowe Bay sand and gravel were with similar proportions, except for a slight correction for the difference in specific gravity in order that the ratios of absolute volumes might be the same. In the earlier tests, mechanical analysis mixtures were made to definite curves, such

as parabolas and straight lines and curves intermediate between these, in order to select the curve giving the maximum density. In the later tests, in the winter of 1904-05, the methods were changed slightly, and the best results were obtained by making mixtures on trial curves without reference to their mathematical equations, and then, having found the best curves, equations were fitted to them, so that they could be more easily applied to different materials and more readily plotted.

All the earlier density specimens were found to be inferior to the tests made at Little Falls. Subsequent results in the manufacture of the beams indicated that this was due to the quality of the cement, reference to which has already been made. From the volumetric tests, and from tests of the composition of beams of neat cement, it appeared that the difference in the density was due to the cement taking so much water in gauging that the volume of the paste was increased, and the density lowered. In the winter of 1904-05 the experiments were continued with the cement which was then being furnished, and this gave satisfactory results. To avoid duplication of tests, and reach conclusions as speedily as possible, the first series of experiments, in the winter of 1904-05, was made with one class of material, broken stone and screenings excavated from Jerome Park Reservoir site, and with 10% of cement, by weight, of total dry material. This was followed by tests with Cowe Bay gravel and sand, and with other percentages of cement.

Necessity for Using Cement in Density Tests.—The necessity may be questioned for using cement in the tests for density, which were really for the purpose of determining the best proportions of the various sizes of particles of the aggregate. Why would it not have been simpler to use only the dry aggregate with no cement or water, and thus more readily obtain the mixtures which would give the least volume with the same weight?

As a matter of fact, both theory and experiment prove that the mixtures of aggregate which give the greatest density when dry do not necessarily give the greatest density when mixed with the cement and water. The cement and water actually occupy space in the mass, as many of the voids are too small for the grains of cement to fit into them without expanding the volume, and the water surrounds the grains of fine sand and cement, and actually increases

the bulk. As an illustration of this, the weight per cubic foot of very fine sand when loose, if weighed absolutely dry, is very nearly the same as the weight per cubic foot of a very coarse sand weighed dry. However, if the two sands are mixed with cement and water, the resulting mortar made with the fine sand will occupy a bulk perhaps 20% greater than the mortar of coarse sand, even when each of them is mixed with the cement in the same proportion by weight or by absolutely dry volume. The density of the mortar of fine sand will be correspondingly less than the mortar of coarse sand. Furthermore, the proportion of cement to sand affects the relative bulk and density of the two mortars, 1 : 1 mixtures giving different comparative results from 1 : 4 mortars.

If fine aggregate, having grains of the same size as cement particles, were used, the aggregate could have been mixed with water without using any cement, and the resulting density would probably have been the same as where real cement replaced the fine aggregate. However, this fine aggregate is more costly than cement because of the labor required to screen it. Moreover, the conditions would not have been so practical as where cement itself is used.

Density Tests with 2¼-in. Jerome Park Stone.—In all these tests, the cement was included in the mechanical analysis curve. This was in accordance with the assumption, which was afterward proved to be correct, that the grains of cement acted similarly to grains of sand of similar size as far as the density was concerned. A trial mechanical analysis curve was drawn, based on previous tests, and a volumetric test was made to determine the density. The curve was then altered in various ways by raising and lowering it at different diameters of stone, and volumetric tests were made with each experimental curve. By this means the general principles of the density of mixtures of Jerome Park broken stone and screenings with 10% Giant Portland cement were studied.

The curve which gave the best result when using a graded coarse aggregate with the cement was found to be one resembling a parabola in appearance, but, more strictly, consisting of a curve having for the lower portion the form of an ellipse, and above this a straight line running to 100% on the maximum diameter of the stone, in this case 2¼ in. The tangent point of the curve and straight line was at about 0.2 in. diameter. The curve started below measurable

diameters on the 7% line, indicating that at least 7% by weight of the very finest diameters of particles of cement or sand, or both, was required for a dense mixture.

In studying these density curves, the attention of the writers was called to experiments by Mr. A. E. Schutté, for the Warren Brothers Company, of Boston, on mixtures of aggregate to be used in their bituminous macadam pavement. For this class of work, which is really a scientifically graded concrete with bitumen for the matrix instead of cement, Mr. Schutté found the densest mixtures, and the best results in practice, to occur when a large percentage, about 50% in fact, of the aggregate consisted of stone of the coarsest diameter and of uniform size. Density tests, using a mixture of this kind, were made at Jerome Park for comparison with the tests with graded coarse stone. The resulting concrete was found to be slightly denser than the concrete with a graded stone. However, the mixture did not look as well in the mixing pan, and while, with a plastic substance like bitumen, a thorough mixture would without doubt produce excellent results, with the cement there seemed to be more of a tendency of the stones to separate from the mortar than with the concrete containing graded coarse stone. Whether or not these conclusions would apply in practice has not yet been determined. Tests of beams made with the two kinds of mixtures, although somewhat erratic, indicated a scarcely appreciable difference in the strength. Under some conditions, a stone of uniform size is as easy to obtain as a graded stone or "crusher run"; hence further experiments are desirable in order to compare these two methods of mixture, and prove whether, under some conditions, the uniform stone may not be economical. The equations of the curve which was selected as the best are given in succeeding paragraphs.

Density Tests with 1-in. and ½-in. Stone.—The best analysis curve for cement and aggregates the maximum size of which was less than 2½ in. was next studied in a similar fashion, using, as before, 10% of cement to the weight of total dry materials. The curves for the smaller stone were found to resemble the 2½-in. curve, except that the tangent began at a smaller diameter. This diameter, where the curve ended and the straight line began, was found to be about one-tenth of the diameter of the maximum size of stone

used in the mixture; thus, for 1-in. stone, the tangent point was at about 0.1 in. diameter, and for ½-in. stone, the tangent point was at about 0.05 in. diameter. This suggested the possibility of a curve for all sizes of stone with the same equation, but with the diameter of the maximum size as a function of one of the terms. It was found that the shape of the curve was not exactly the same for the different sizes, but by introducing a small constant in the values of the axes of the ellipses, an equation was found which fitted all the diameters.

Cement versus Fine Sand.—The experiments just described assume that as far as density is concerned, cement acts in the same way as sand with grains of the same size. The cement, therefore, was included in making up the mechanical analysis curve. To prove this assumption, several other percentages, ranging from 8 to 15%, of cement to the weight of total aggregate, were tried, using the best curve already found for the mixtures with 10% of cement, and the resulting densities were substantially identical with the density of the mixture by the same curve using 10% cement. It is evident from this that correct proportioning for concretes of maximum density, but of different strength, consists in not simply increasing the percentage of cement, if a richer mixture is required, but in substituting more cement for a like absolute volume of sand having grains of the same size as the cement. In other words, the larger the proportion of cement, the fewer very fine grains of sand are required, because the cement takes the place of them in increasing the density.

Density Tests with Cowe Bay Material.—Tests of density of concrete composed of cement with gravel and sand from Cowe Bay were made in a similar manner to those of cement with Jerome Park stone and screenings. This investigation is of interest not only with reference to the work at Jerome Park, but to throw light on the mooted question of the relative value of broken stone and gravel, and form some basis for an economical comparison of the two in any given locality.

Tests were made with concrete composed of straight Cowe Bay material, that is, gravel and sand and cement, and with concrete of Jerome Park broken stone, Cowe Bay sand and cement. The best curves for both of these combinations were found to be similar to

the curves for concrete with straight Jerome Park material, except that, because of their rounded nature, the particles packed more closely together, so that similar mixtures gave with the Cowe Bay material a greater density, and for maximum density a smaller quantity of fine material was required, the curves in the diagram for Cowe Bay material being lower on the ordinate corresponding to one-tenth the maximum diameter of the stone. The combination of Jerome Park stone and Cowe Bay sand required more fine material than the Cowe Bay gravel and sand, but less than the Jerome Park stone and screenings.

Ideal Sand.—The character of the best or ideal sand does not appear to depend upon the shape of the coarse aggregate, the best sand for Jerome Park broken stone also being found best for the Cowe Bay gravel of the same sized grains, although less was required with the latter. On the other hand, the curve for the Jerome Park screenings (plus cement) was slightly different from the curve for Cowe Bay sand (plus cement), showing a different arrangement of grains.

Equations of Ideal Mechanical Analysis Curves of Density.

Having found by trial the curves for the best analyses of each size and class of material, mathematical curves were fitted to them for convenience in plotting. As already stated, all the curves for aggregate plus cement consist of ellipses with straight lines tangent to them. The curves all start upon and are tangent to the vertical zero axis of percentages at 7%—that is, at least 7% of the aggregate plus cement is finer than the No. 200 sieve—and run as ellipses with axes differing with the character of the materials, to a point which was found to be on a vertical ordinate or diameter the value of which is about one-tenth the diameter of the maximum particles of stone, and thence by a tangent to the ordinate of maximum diameter, intersecting this on the 100% abscissa.

The general equation of an ellipse, using its own axes, that is, with zero of both co-ordinates at the center of the ellipse, is

$$y = \frac{b}{a}\sqrt{a^2 - x^2}.$$

This is the simplest equation to use for plotting, as it is in regular form, and only the values of a and b (that is, the semi-major and

semi-minor axes) are required. Using the zero co-ordinates shown on the mechanical analysis diagram, and having the center of the ellipse at a point, $x = a - y = 7$, the equation becomes

$$(y - 7)^2 = \frac{b^2}{a^2}(2\,a\,x - x^2).$$

The values of a and b found for the different materials, including the cement, are as follows:

Materials.	a	b
Jerome Park stone and screenings.....	$0.035 + 0.14\,D$	$29.4 + 2.2\,D$
Cowe Bay gravel and sand.....	$0.04 + 0.16\,D$	$26.4 + 1.3\,D$
Jerome Park stoue and Cowe Bay sand.....	$0.04 + 0.16\,D$	$28.5 + 1.3\,D$

In this table, D = the maximum diameter of the stone, in inches.

It should be remembered that these experiments on density were on small laboratory specimens, thoroughly mixed, and carefully placed and compacted; and the values thus represent ideal conditions which must be modified for working conditions by increasing the value of b and thus adding more sand. Working formulas are given in Table 5.

Beam Tests.

While the experiments to obtain the laws of maximum density were in progress, during 1904, and before the results were available, a series of beams was made up with various proportions of Portland cement, Jerome Park screenings and Jerome Park crusher run stone, for the purpose of educating the men in handling the apparatus and making concrete. In this way 143 beams were made, and their strength and density determined. The results are given in Table 4. Owing to the peculiarity of the cement, to which reference has already been made, the breaking strengths are low, but the relative results agree closely with those at Little Falls.*

As soon as the results of the density tests were available in 1905, a series of beams was made up according to definite plans, to compare the strength of concrete made with different materials and different proportions of the various diameters of particles. Cement and aggregate, graded by curves of mechanical analysis which were found by the volumetric tests to produce concrete of maxi-

* The results at Little Falls may be found in Taylor and Thompson's "Concrete, Plain and Reinforced," pp. 258-9.

mum density for a given kind of material, were used as a basis for the tests. The strengths of these ideal specimens were then compared with the strengths of concrete mixtures proportioned by the artificial mechanical analysis curves which in the volumetric tests showed less density, to establish the general law that with the same percentage of cement the densest concrete is the strongest; the strengths of these ideal mixtures were compared with the strength of mixtures of natural material in ordinary proportions, and also in the best possible natural proportions; the relative strengths of concretes made with stone of different maximum sizes were compared; the relative strengths of concrete made with Jerome Park stone and screenings were compared with Cowe Bay gravel and sand and with Jerome Park stone and Cowe Bay sand; and the comparative strengths of neat cement and concrete beams made with different brands of cement were tested.

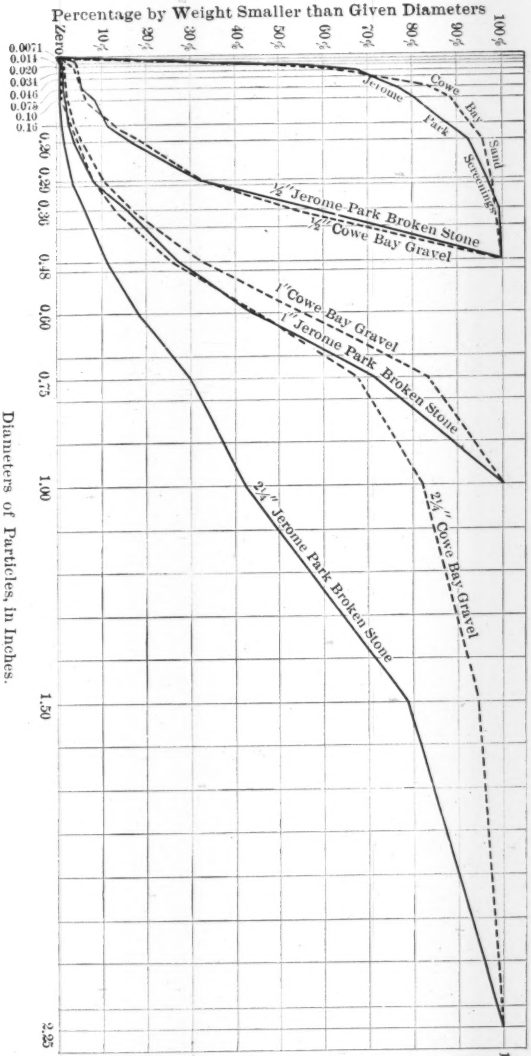
It was intended that two beams should be made from each mixture; each beam being manufactured separately, however, in order that the exact weights of materials entering into it could be determined. In certain cases it was found expedient to make only one beam of a mixture, but the results with these specimens were not as good, three transverse breaks and two compressive breaks not being sufficient to give a fair average of the material. The full list of tests, of which there were 116 specimens in all, are briefly scheduled as follows:

Straight Jerome Park material, using, respectively, 2¼-in., 1-in. and ½-in. stone, mixed with screenings and 10% cement, by weight, to total dry materials, and graded to the best ideal curves for each size, and to other artificial curves having more sand and less stone than the ideal.

Straight Jerome Park material, using, respectively, 2¼-in., 1-in. and ½-in. stone, mixed with screenings and cement in three natural proportions—1 : 2½ : 6½, 1 : 3 : 6, and 1 : 3½ : 5½.

Straight Jerome Park material, using for the coarse aggregate, stone of uniform size, 2¼-in., 1-in. and ½-in. diameter, respectively, mixed with screenings and 10% of cement, by weight, this fine material being graded to an ellipse.

Straight Jerome Park material, 2¼-in. stone and screenings, mixed to the best ideal curve with 8, 12½ and 15% cement to total weight of dry materials.



AVERAGE MECHANICAL ANALYSES OF VARIOUS NATURAL MATERIALS.

FIG. 4.

TABLE 4 (*Continued*).

3.4	1:1 :2	34.0	34.0	68.2	136.2	15.6	151.8	138.9	151.2	0.175	0.192	0.392	0.584	0.759	0.250	1.009	0.186	0.055	0.241	112	6	550	385	479
7.8	1:1 :3.5	25.6	25.6	89.6	140.8	13.3	154.1	142.8	153.8	0.132	0.145	0.515	0.660	0.792	0.213	1.005	0.141	0.067	0.208	112	6	562	323	433
35.36	1:1 :5	20.8	20.8	104.1	145.7	12.8	158.5	147.4	156.7	0 107	0.118	0.598	0.716	0.823	0.205	1.028	0.114	0.063	0.177	112	6	315	218	278
84.85	1:2 :0	36.6	73.0	......	109.6	26.3	135.9	112.5	134.5	0.189	0.412	......	0.412	0.601	0.422	1.023	0.201	0.198	0.399	112	6	500	437	461
47.48	1:2 :1.5	28.2	56.4	42.5	127.1	18.2	145.3	129.4	145.3	0.145	0.319	0.244	0.563	0.708	0.292	1.000	0.154	0.138	0.292	112	6	394	313	350
9.10	1:2 :3	20.2	40.4	60.5	121.1	13.3	134.4	122.7	141.1	0.104	0.228	0.348	0.576	0.680	0.213	0.893	0.111	0.209	0.320	112	6	378	133	307
19.20	1:2 :5	17.7	35.4	88.5	141.6	13.1	154.7	143.0	154.1	0.091	0.200	0.509	0.709	0.800	0.210	1.010	0.097	0.103	0.200	112	4	276	233	253
53.54	1:2 :5	17.6	35.2	87.6	140.4	13.2	153.6	141.8	153.3	0.091	0.199	0.503	0.702	0.793	0.212	1.005	0.097	0.110	0.207	112	4	216	134	184
135.136	1:2 :5	17.6	35.2	87.9	140.7	14.3	155.0	142.1	153.5	0.091	0.199	0 505	0.704	0.795	0.229	1.024	0.097	0.108	0.205	120	5	354	222	308
37.38	1:2 :7.5	13.8	27.6	103.8	145.2	11.9	157.1	146.3	156.2	0.071	0.156	0.597	0.753	0.824	0.191	1.015	0.076	0.100	0.176	112	4	315	197	249
69	1:3 :0	28.2	84.6	......	112.8	23.4	136.2	115.1	136.3	0.145	0.478	......	0.478	0.623	0.375	0.998	0.154	0.223	0.377	56	3	281	254	271
70	1:3 :0	....	....	......	......	....	......	......	......	......	......	......	......	......	......	......	......	......	......	28	2	242	211	226
121.122	1:3 :0	27.4	82.4	......	109.8	24.5	134.3	112.0	134.3	0.141	0.466	......	0.466	0.607	0.393	1.000	0.150	0.243	0.393	84	6	324	292	306
49.50	1:3 :2	21.1	63.4	42.2	126.7	18.6	145.3	128.4	144.8	0.169	0.358	0.243	0.601	0.710	0.298	1.008	0.116	0.174	0.290	112	3	239	222	231
139.140	1:3 :4	16.7	50.0	66.7	133.4	17.0	150.4	134.7	148.9	0.086	0.282	0.383	0.665	0.751	0.273	1.024	0.092	0.157	0.249	120	5	304	190	254
15.16	1:3 :4	17.2	51.6	68.9	137.7	15.8	153.0	139.1	151.6	0.089	0.292	0.396	0.686	0.777	0.245	1.022	0.095	0.128	0.223	112	2	220	151	185
27.28	1:3 :4	17.1	51.3	68.4	136.8	15.1	151.9	138.2	151.1	0.088	0.290	0.393	0.683	0.771	0.242	1.013	0.094	0.135	0.229	112	1	130	130	130
51.52	1:3 :4	17.2	51.5	68.6	137.3	15.7	153.0	138.7	151.4	0.089	0.291	0.394	0.685	0.774	0.252	1.026	0.095	0.131	0.226	112	3	222	112	164
111	1:3 :4	16.4	49.1	65.5	131.0	16.9	147.9	132.3	147.3	0.085	0.277	0.376	0.653	0.738	0.271	1.009	0.091	0.171	0.262	112	2	211	135	173
127.128	1:3 :6	13.7	41.1	82.0	136.8	15.7	152.5	137.9	150.9	0.071	0.232	0.471	0.703	0.774	0.252	1.026	0.076	0.150	0.226	84	4	199	141	170
78	1:3 :6	13.9	41.7	83.4	139.0	15.4	154.4	140.1	152.3	0.072	0.236	0.479	0.715	0.787	0.247	1.034	0.077	0.136	0.213	56	2	90	76	83
79	1:3 :6	....	....	......	......	....	......	......	......	......	......	......	......	......	......	......	......	......	......	28	2	99	96	98
21.22	1:3 :7	12.9	39.0	90.8	142.7	13.1	155.8	143.7	154.6	0.067	0.220	0.522	0.742	0.809	0.210	1.019	0.071	0.120	0.191	112	4	177	147	160
55.56	1:3 :7	13.0	39.0	90.9	142.9	13.6	156.5	143.9	154.8	0.067	0.220	0.522	0.742	0.809	0.218	1.027	0.071	0.120	0.191	112	2	192	181	187
88.89	1:4 :0	21.9	87.4	......	109.3	25.4	134.7	111.1	133.8	0.113	0.494	......	0.494	0.607	0.407	1.014	0.120	0.273	0.393	112	6	274	209	225
94.95	1:4 :3	15.8	63.3	47.4	126.5	18.9	145.4	127.8	144.5	0.081	0.358	0.272	0.630	0.711	0.303	1.014	0.086	0.203	0.289	112	3	221	189	203
31.32	1:4 :6	12.5	50.2	75.3	138.0	15.1	153.1	139.0	151.7	0.064	0.284	0.433	0.717	0.781	0.242	1.023	0.068	0.151	0.219	112	5	166	120	144
118.119	1:4 :6	12.2	48.8	73.2	134.2	15.8	150.0	135.2	149.2	0.063	0.276	0.421	0.697	0.760	0.253	1.013	0.067	0.173	0.240	111	3	222	125	178
5.6	1:4 :6	12.6	50.4	75.4	138.4	15.3	153.7	139.4	151.9	0.065	0.285	0.433	0.718	0.783	0.245	1.028	0.069	0.148	0.217	112	2	156	121	138
23.24	1:4 :9	10.0	40.0	90.0	140.0	15.7	155.7	140.8	152.8	0.052	0.226	0.517	0.743	0.795	0.252	1.047	0.055	0.150	0.205	112	1	63	63	63
57.58	1:4 :9	10.3	41.4	93.0	144.7	13.7	158.4	145.5	155.9	0.053	0.234	0.534	0.768	0.821	0.220	1.041	0.056	0.123	0.179	112	1	154	154	154
116.117	1:4 :9	10.0	39.9	89.7	139.6	15.1	151.7	140.4	152.5	0.052	0.225	0.516	0.741	0.793	0.242	1.035	0.055	0.152	0.207	111	1	128	128	128

TABLE 4.—Composition and Transverse Strength of Concrete Beams made of Giant Portland Cement, Jerome Park Screenings, and Run of Crusher Jerome Park Stone, $\frac{1}{10}$ Inch to $2\frac{1}{4}$ Inches in Diameter.

Beams, 6 by 6 by 72 in. Spans, 30 and 60 in.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)
		Weight, in Pounds, of Material in 1 Cu. Ft. of Beam, as Mixed.								Calculated Volume, in Cubic Feet, of Material in 1 Cu. Ft. of Beam, as Mixed.							Volume of Voids in 1 Cu. Ft.			Modulus of Rupture.				
							Totals.															Pounds per square inch.		
Reference numbers.	Proportions, by weight. C. S. G.	Cement.	Sand.	Stone.	Total dry mix.	Water.	As mixed.	Minimum.	Maximum.	Cement.	Sand.	Stone.	Total sand and stone.	Total dry.	Water.	Total.	Cement.	Aggregate.	Total.	Age, in days.	No. of breaks.	Maximum.	Minimum.	Average.
73	1:0 :0	93.7	....	.....	93.7	31.2	124.9	101.2	125.9	0.483	.....	.....	.....	0.483	0.500	0.983	0.517	.....	0.517	83	3	518	386	442
120	1:0 :0	93.5	....	.....	93.5	31.8	125.3	101.0	125.7	0.482	.....	.....	.....	0.482	0.510	0.992	0.518	.....	0.518	111	3	917	896	906
143	1:0 :0	94.7	....	.....	94.7	32.3	127.0	102.3	126.9	0.488	.....	.....	.....	0.488	0.518	1.006	0.512	.....	0.512	119	3	963	935	946
13,14	1:0 :0.8	67.6	....	54.1	121.7	20.5	142.2	127.1	143.0	0.348	.....	0.311	0.311	0.659	0.329	0.988	0.371	0.030	0.341	112	6	664	570	629
17,18	1:0 :1.7	49.6	....	84.2	133.8	16.7	150.5	137.8	150.0	0.256	.....	0.484	0.484	0.740	0.268	1.008	0.273	0.013	0.260	112	6	652	492	556
29(	1:0 :2.5	40.6	....	101.3	141.9	14.5	156.4	145.1	154.9	0.209	.....	0.582	0.582	0.791	0.232	1.023	0.223	0.014	0.209	56	3	384	363	371
30(	1:0 :2.5	....	....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	28	2	357	300	329
125,126	1:0 :2.5	39.4	....	98.4	137.8	15.2	153.0	141.0	152.2	0.203	.....	0.556	0.556	0.769	0.244	1.013	0.216	0.015	0.231	84	6	562	415	461
43,44	1:0.5:0.5	59.6	29.8	29.8	119.2	21.6	140.8	124.0	141.3	0.307	0.168	0.171	0.339	0.646	0.346	0.992	0.327	0.027	0.354	112	6	618	547	579
33,34	1:0.5:3.7	27.9	14.0	103.3	145.2	12.7	157.9	147.4	156.6	0.144	0.079	0.594	0.673	0.817	0.204	1.021	0.153	0.030	0.183	112	6	411	338	386
82,83	1:1 :0	54.6	54.6	.....	109.2	25.5	134.7	113.6	134.8	0.281	0.308	.....	0.308	0.589	0.409	0.998	0.299	0.112	0.411	112	6	612	553	586
45,46	1:1 :1	42.5	42.5	42.5	127.5	18.4	145.9	130.9	146.0	0.219	0.240	0.244	0.484	0.703	0.295	0.998	0.233	0.064	0.297	112	6	522	437	484

TABLE 4 (*Continued*).

(1)	(2)			(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)
				Weight, in Pounds, of Material in 1 Cu. Ft. of Beam, as Mixed.								Calculated Volume, in Cubic Feet, of Material in 1 Cu. Ft. of Beam, as Mixed.							Volume of Voids in 1 Cu. Ft.			Modulus of Rupture.				
Reference numbers.	Proportions, by weight.			Cement.	Sand.	Stone.	Total dry mix.	Water.	Totals.			Cement.	Sand.	Stone.	Total sand and stone.	Total dry.	Water.	Total.	Cement.	Aggregate.	Total.	Age, in days.	No. of breaks.	Pounds per square inch.		
	C.	S.	G.						As mixed.	Minimum.	Maximum.													Maximum.	Minimum.	Average.
90,91	1	5	0	18.6	93.0	.....	111.6	23.2	134.8	113.1	135.2	0.096	0.525	.....	0.525	0.621	0.372	0.993	0.102	0.277	0.379	112	3	211	115	164
76	1	6	0	15.5	93.4	.....	108.9	26.1	135.0	110.1	133.4	0.080	0.528	.....	0.528	0.608	0.418	1.026	0.085	0.307	0.392	56	3	170	95	125
77	.....			.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	28	2	82	48	65
123,124	1	6	0	15.4	92.5	.....	107.9	25.2	133.1	109.1	132.7	0.079	0.523	.....	0.523	0.602	0.404	1.006	0.084	0.314	0.398	84	4	208	145	175
11,12	1	6	7	9.3	56.0	65.3	130.6	15.5	146.1	131.3	146.9	0.048	0.316	0.375	0.691	0.739	0.248	0.987	0.051	0.210	0.261	112	3	89	39	66
59,60	1	6	7	9.6	57.5	67.2	134.3	16.4	150.7	135.0	149.2	0.050	0.325	0.386	0.711	0.761	0.263	1.024	0.053	0.186	0.239	112	1	114	114	114
112,113	1	6	7	9.4	56.4	65.8	131.6	17.8	149.4	132.4	147.5	0.049	0.319	0.378	0.697	0.746	0.285	1.031	0.052	0.202	0.254	112	4	116	88	104
65	1	6	12	7.5	45.1	90.3	142.9	14.7	157.6	143.5	154.6	0.039	0.255	0.519	0.774	0.813	0.236	1.049	0.042	0.145	0.187	56	1	50	50	50
66	.....			.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	28	2	58	51	55
25,26	1	6	12	7.6	45.8	91.6	145.0	14.3	159.3	145.6	156.0	0.039	0.259	0.526	0.785	0.824	0.229	1.053	0.042	0.134	0.176	112	1	72	72	72
114,115	1	6	12	7.2	43.5	86.8	137.5	15.1	152.6	138.1	151.1	0.037	0.246	0.499	0.745	0.782	0.242	1.024	0.039	0.179	0.218	111	1	115	115	115
107,108	1	8	0	11.9	95.4	.....	107.3	26.4	133.7	108.2	132.3	0.061	0.539	.....	0.539	0.600	0.423	1.023	0.065	0.335	0.400	112	4	162	73	127

Volumes, in cubic feet per 100 lb. as mixed: Cement, 1.00; Sand, 1.06; Stone, 1.03. Specific Gravity: Sand, 2.81; Stone, 2.785; Cement, 3.09. Weights, in pounds per cubic foot, as mixed: Cement, 100; Sand, 94.8; Stone, 97.0; Water, 62.4.

Tensile strength of cement, in pounds per square inch, neat: 7 days, 536; 28 days, 574; 6 months, 688.

Mechanical analyses; percentage by weight of particles, below diameter: Sand, 100%, 0.60; 93%, 0.20; 81%, 0.10; 66%, 0.34; 48%, 0.016; 21%, 0.0071; 2%, 0.0027. Stone, 100%, 2.25; 42%, 1.00; 11%, 0.48; 1%, 0.10.

Column 9 = Column 3 × 0.08 + Column 6. Column 10 = Column 6 + (Column 20 × 62.4). Column 9 represents the theoretical minimum, assuming 8% of water for chemical combination. Column 10 represents the theoretical maximum, assuming that the voids are filled with water.

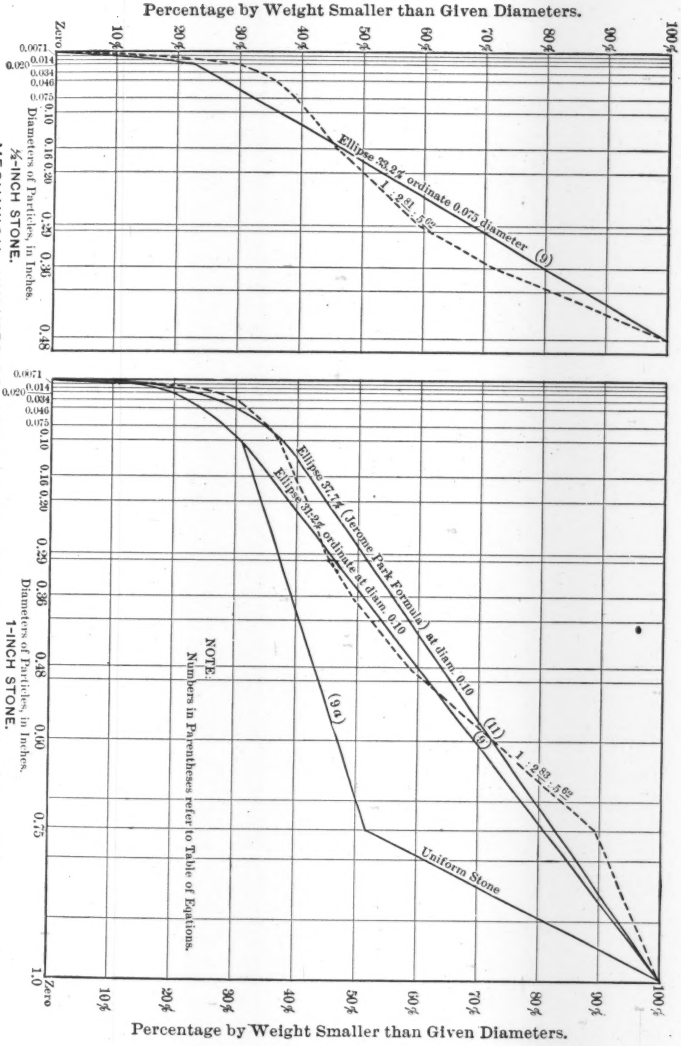

Percentage by Weight Smaller than Given Diameters.
Zero 10% 20% 30% 40% 50% 60% 70% 80% 90% 100%
0.0071 0.014 0.020 0.034 0.046 0.075 0.10 0.16 0.20 0.29 0.35 0.36 0.48
Diameters of Particles, in Inches.
¼-INCH STONE.
Ellipse 33.3% ordinate 0.075 diameter (9)
1 : 2.21 : 5.62
MECHANICAL ANALYSES OF MIXTURES OF CEMENT, COWE BAY SAND, AND STONE OF ¼ IN. AND 1 IN. MAXIMUM DIAMETER USED IN BEAMS.
0.0071 0.014 0.020 0.034 0.046 0.075 0.10 0.16 0.20 0.29 0.35 0.36 0.48 0.60 0.75 1.0
Diameters of Particles, in Inches.
1-INCH STONE.
Ellipse 37.7% (Jerome Park Formula) at diam. 0.10
Ellipse 31.2% ordinate at diam. 0.10
(9a)
(11)
(6)
1 : 2.83 : 5.62
Uniform Stone
NOTE:
Numbers in Parentheses refer to Table of Equations.
Zero 10% 20% 30% 40% 50% 60% 70% 80% 90% 100%
Percentage by Weight Smaller than Given Diameters.

Fig. 5.

Straight Cowe Bay material, using, respectively, 2½-in., 1-in. and ½-in. gravel, mixed with Cowe Bay sand and 10% cement, by weight, and graded to the best ideal curve for each size; also, in natural proportions, 1 : 3 : 6; and also to a curve identical with one of the straight Jerome Park curves.

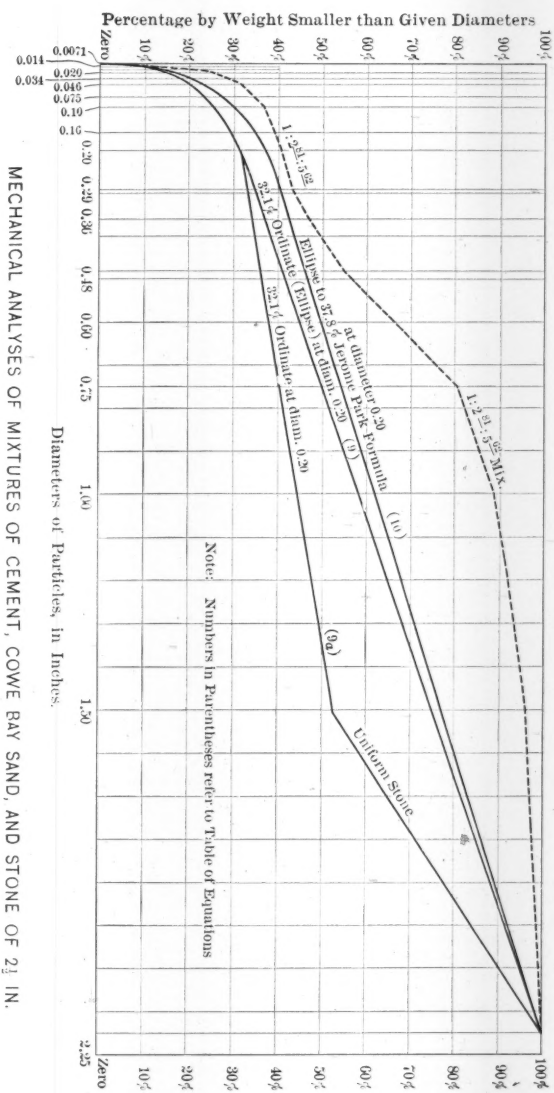
Straight Cowe Bay material, using 2½-in. and 1-in. stone of uniform size, mixed with Cowe Bay sand and 10% cement, by weight, and this fine material graded to an ellipse.

Mixed Jerome Park broken stone and Cowe Bay sand, using, respectively, 2½-in., 1-in. and ½-in. stone, mixed with Cowe Bay sand, and 10% cement, by weight, and graded to the best ideal curve for each size; also, in natural proportions, 1 : 3 : 6; and also to a curve identical with one of the straight Jerome Park curves.

Methods of Preparing Materials for Natural Proportions.—A few mixtures were made in ordinary proportions, 1 : 2½ : 6½, 1 : 3 : 6, and 1 : 3½ : 5½, as scheduled previously, for comparison with the ideal proportions. Instead of using for the aggregates the natural materials as they came from the crusher, which varied from day to day because of the difference in methods of handling and in the character of the stone in different parts of the ledge, an average mechanical analysis of the screenings as they came from the crusher was made by averaging a number of analyses of different lots of screenings. Average analyses of Jerome Park broken stone, of Cowe Bay gravel, and of Cowe Bay sand, were similarly found. These average analyses are shown in Fig. 4. The analyses of the two materials which were to be used together were plotted on a diagram and combined in the required proportions by the ordinary method of combining mechanical analysis curves.

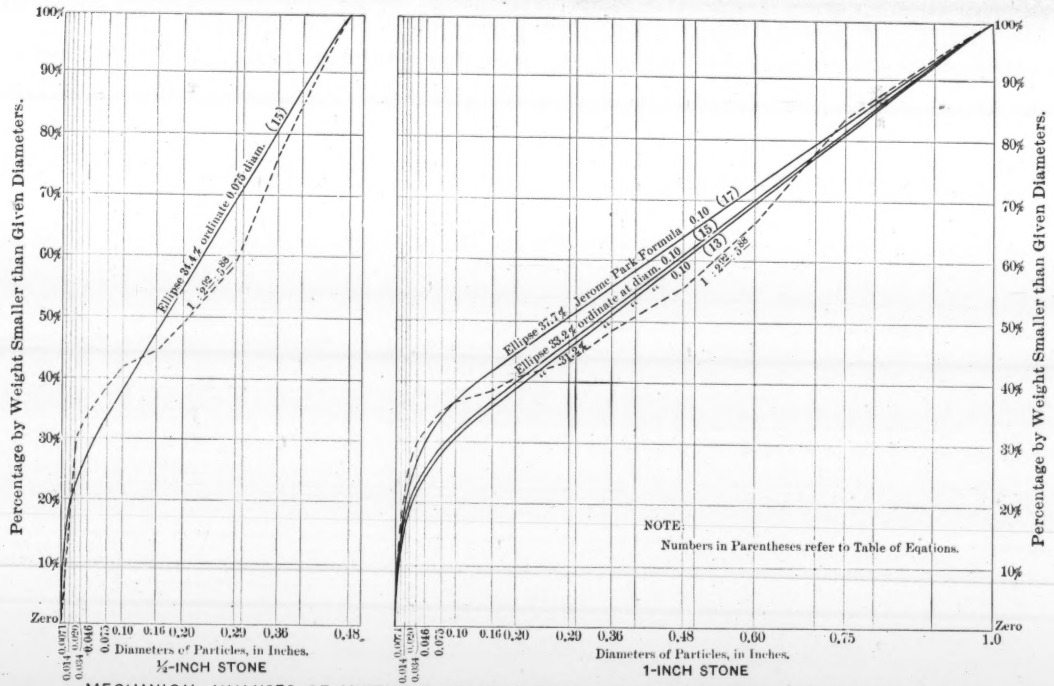
The weights of the various-sized particles for each of the beams, which were graded to so-called natural mixtures, were obtained from these combined curves. This eliminated the variation in the materials coming from the crusher and from the dredge on different days, and gave truly average mixtures.

Mechanical Analysis Curves Used.—All the mechanical analyses of the aggregate and cement which were used in the concretes for the 1905 beam tests are drawn to scale in Figs. 5 to 10, inclusive, and Table 5 gives in full the equations of all the curves of artificial mixes which were used. The curves are numbered, and correspond-



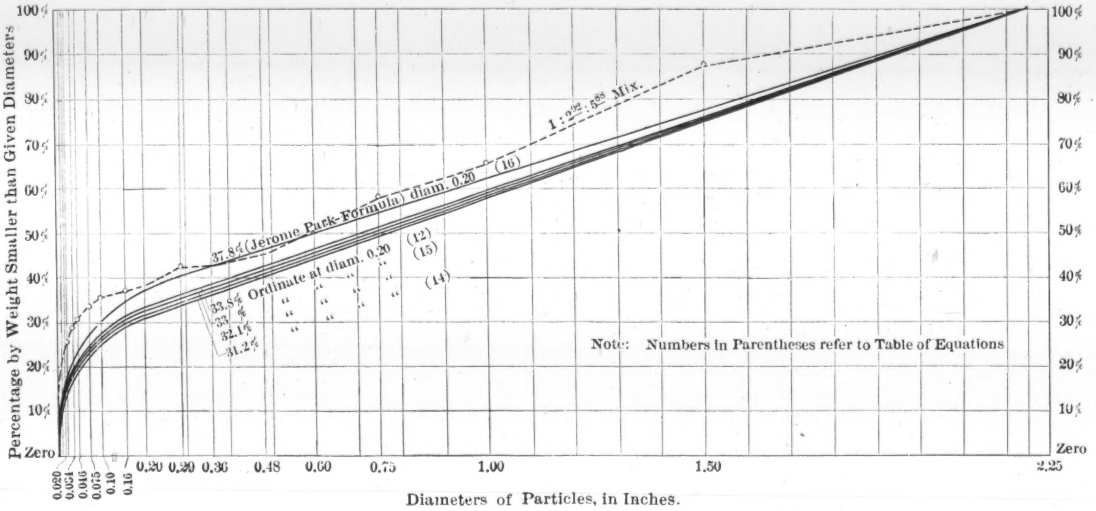
MECHANICAL ANALYSES OF MIXTURES OF CEMENT, COWE BAY SAND, AND STONE OF 2½ IN. MAXIMUM DIAMETER USED IN BEAMS.

FIG. 6.



MECHANICAL ANALYSES OF MIXTURES OF CEMENT, COWE BAY SAND, AND JEROME PARK STONE OF ½ AND 1 IN. MAXIMUM DIAMETER USED IN BEAMS.

FIG. 7.



MECHANICAL ANALYSES OF MIXTURES OF CEMENT, COWE BAY SAND, AND JEROME PARK STONE OF 2¼ IN. MAXIMUM DIAMETER USED IN BEAMS.

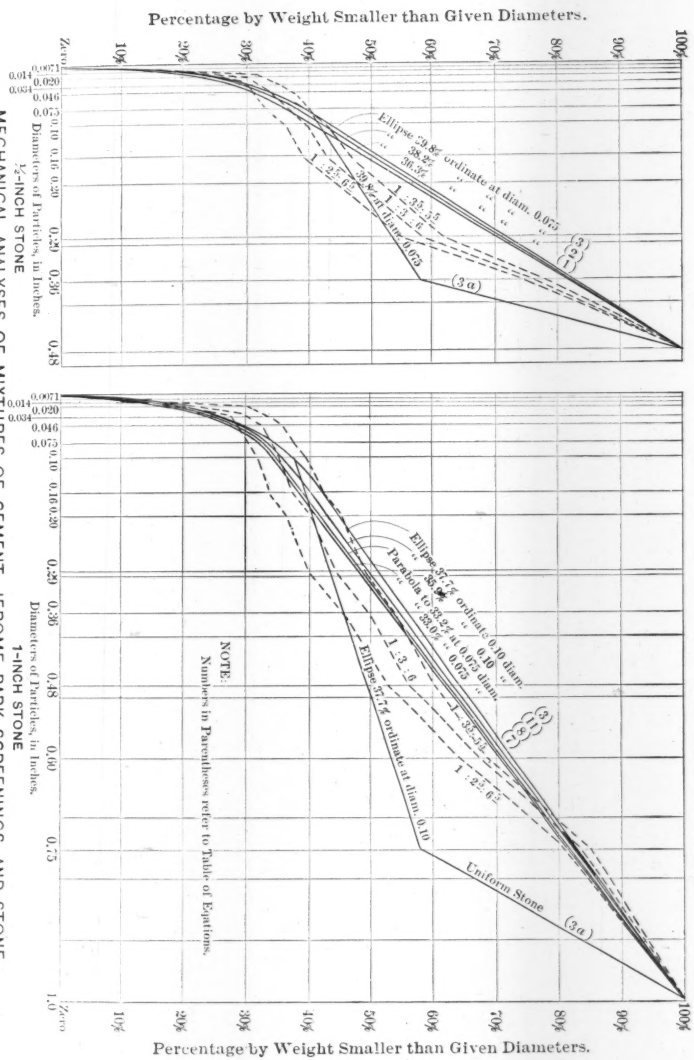
FIG. 8.

ing numbers are placed on the curves in the diagrams for convenience of reference.

It was found by experiment that more sand was required in the curves used in the beam mixtures than was indicated by the best ideal curves in the volumetric tests for density. Accordingly, the curves in the diagrams are all uniformly higher than would be expected from the density experiments. In some of the diagrams, a larger number of curves were drawn than would appear from the schedule to be needed. This was due to the fact just mentioned. The first mixtures were made with curves corresponding to the best curves of the density tests, and it was found necessary to raise all of them about 2% at the tangent point to prevent a very rough surface to the beam, which indicated that there was not sufficient mortar to fill the voids in the stone. In the table, which gives the full data of the beam tests, these specimens are designated in the remarks. The results from them, especially in the density tests, cannot be considered as entirely reliable.

Methods and Apparatus.—The method of weighing the materials for the beams and mixing them was similar to the processes in the volumetric tests for density just described. The tools and various implements used in making the beams are shown in Fig. 2, Plate X.

Weighing.—The weight of each diameter of aggregate and of cement was calculated by direct proportion from the percentage curve, and the approximate quantity of water to be used was estimated from the quantity used in the volumetric tests made with materials of the same mechanical analysis. The total quantity of dry materials varied appreciably with the maximum size of stone and with the character of the material. The larger the stone, the greater the weight of material required for a beam, because of the greater density. The Cowe Bay material has lower specific gravity than the Jerome Park stone and screenings, so that a less weight was required on this account, but the density of the resulting concrete was greater, so that this nearly balanced the other. The quantity for a mixture with 1 : 9 proportions by weight averaged about 220 lb. of aggregate and 25 lb. of cement. The weights actually used for each beam may be found directly from the data in Table 8.



MECHANICAL ANALYSES OF MIXTURES OF CEMENT, JEROME PARK SCREENINGS, AND STONE OF ½ AND 1 IN. MAXIMUM DIAMETER USED IN BEAMS.

FIG. 9.

JEROME PARK SCREENINGS AND STONE GRADED TO PARABOLAS AND STRAIGHT LINE.

(7)	144, 145.	.....	$\frac{\sqrt[3]{D}}{29}$	....	(22 9)	.....	(30)	.....	.....	Special curve (abandoned).
(8)	146, 147.	.....	$\frac{\sqrt[3]{D}}{32}$	....	(24.4)	.....	(32)	.....	.....	Special curve (abandoned).

COWE BAY SAND AND STONE GRADED TO ELLIPSE AND TANGENT STRAIGHT LINE.

(9)	217, 229, 223, 226, 259, 224, 225, 233.	$0.04 + 0.16\ D$	$26.4 + 1.3\ D$	0.40	(29)	0.20	(28)	0.12	27	Ideal curve.
(10)	234.	$0.035 + 0.14\ D$	$29.4 + 2.2\ D$	0.35	(34)	.....	.....	.....	.....	Like curve (1)
(11)	235.	$0.035 + 0.14\ D$	$31.4 + 2.2\ D$	....	.....	0.175	(34)	.....	.....	Like curve (3)

JEROME PARK SCREENINGS AND COWE BAY STONE GRADED TO ELLIPSE AND STRAIGHT LINE.

(12)	213, 214, 216, 218.	$0.04 + 0.16\ D$	$28.5 + 1.3\ D$	0.40	(31)	0. 20	(30)	0.12	29	Ideal curve.
(13)	201, 202.	$0.04 + 0.16\ D$	$26.5 + 1.3\ D$	....	(28)	0. 20	(28)	.....	.....	
(14)	200.	$0.04 + 0.16\ D$	$25.5 + 1.3\ D$	0.40	.....	.....	.....	.....	.....	
(15)	209, 210, 211, 212.	$0.04 + 0.16\ D$	$27.5 + 1.3\ D$	0.40	(30)	0. 20	(29)	0.12	28	
(16)	219.	$0.035 + 0.14\ D$	$29.4 + 2.2\ D$	0.35	(34)	.....	.....	.....	.....	Like curve (1)
(17)	215.	$0.035 + 0.14\ D$	$31.4 + 2\ 2\ D$	....	.....	0.175	(34)	.....	.....	Like curve (3)

TABLE 5.—EQUATIONS OF MECHANICAL ANALYSIS CURVES FOR PROPORTIONING CONCRETE BEAMS, IN 1905 TESTS.

Ellipses: (1), $(y - 7)^2 = \frac{b^2}{a^2}(2\,a\,x - x^2)$. (2), $y = + \frac{b}{a}\sqrt{a^2 - x^2}$.

Straight Line: $y = \frac{100 - y^1}{D - x^1}(x - x^1) + y^1$.

Parabola: $y = 7 + 100\sqrt{b\,x}$

a and b are axes of ellipses and parabolas, horizontal and vertical.

D is the diameter of the maximum-sized stone, in inches.

Curve number (see diagram).	Reference numbers of beams.	General Values.		Numerical Values.						Remarks.
				$D = 2\frac{1}{4}$.		$D = 1$.		$D = \frac{1}{2}$.		
		a	b	a	b	a	b	a	b	
	JEROME PARK SCREENINGS AND STONE GRADED TO ELLIPSE AND TANGENT STRAIGHT LINE.									
(1)	155, 157, 151, 152, 164, 165, 179, 180, 183, 184, 193.	$0.035 + 0.14\,D$	$29.4 + 2.2\,D$	0.35	(34)	0.175	(32)	0.105	(30)	Ideal curve.
(2)	148, 153, 154, 178.	$0.035 + 0.14\,D$	$27.4 + 2.2\,D$	0.35	(32)	.....	.....	0.105	(32)	
(3)	156, 158, 195, 196, 159, 160, 181, 182, 185, 186, 197.	$0.035 + 0.14\,D$	$31.4 + 2.2\,D$	0.35	(36)	0.175	(34)	0.105	(34)	
(4)	189, 190, 191.	$0.035 + 0.14\,D$	$33.4 + 2.2\,D$	0.35	(38)	.....	.....	.....	.....	
(5)	149.	.....	.....	0.45	(36.2)	.....	.....	.....	.....	Special curve (abandoned).
(6)	150.	.....	.....	0.45	(38.2)	.....	.....	.....	.....	Special curve (abandoned).

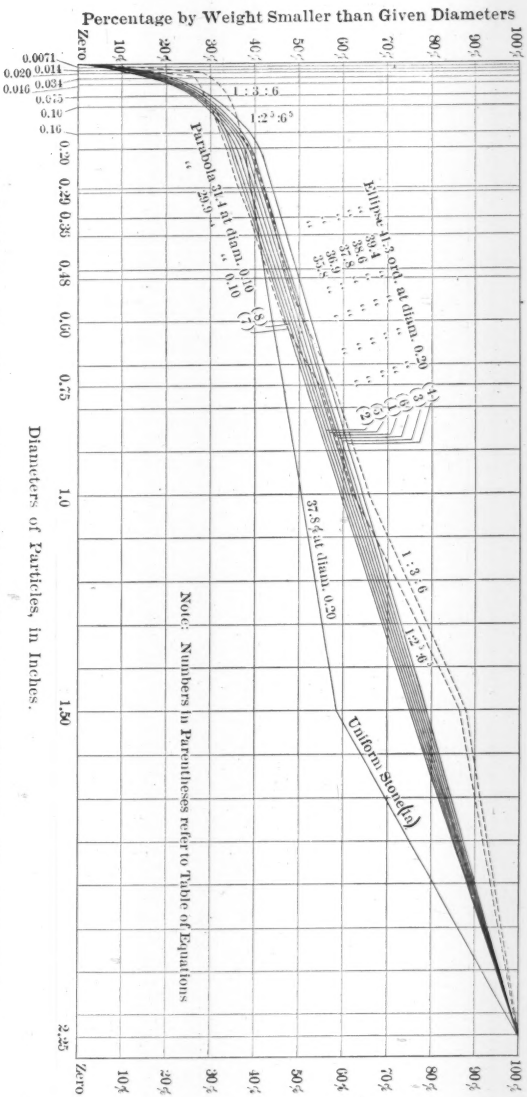
The can for receiving the material, about 18 in. in diameter and 2 ft. deep, was placed on the scale and the tare was recorded by the weigher. Beginning with the finest, two men scooped the different sizes of aggregates, and also the cement, into the can, the poise being slid ahead for each size.

Mixing.—The dry material was then dumped upon the 5 by 8-ft. mixing platform, which was made of a plate of sheet iron surrounded by a strip of wood to prevent the soft material from overflowing. Four men turned the dry material with square-pointed shovels, just as hand-mixed concrete is turned in practice. Three turnings were given, dry. Then the mixture was formed into a circle, and the approximate quantity of water was weighed and turned into it. The material was then mixed wet by the four men, as in practice. Three turnings were given to it, to insure a uniform mix throughout the beam.

Consistency.—The required consistency was soft and mushy, but not wet enough for the mortar to run away from the stones, scoop shovels being necessary to handle the wet concrete. If the calculated weight of water was not sufficient to give the required consistency, more was added, and its weight recorded.

Marks for Specimens.—Four marks were embedded in each beam. The tags consisted of pieces of brass stamped by hand by a die, with the number of the beam followed by the letters, *A, B, C* and *D,* respectively. Each tag was about 1½ in. long by ½ in. wide, and had projections bent up from it to run into the concrete. They were definitely located in the bottom of the mould by measurement, so as to lie nearly, but not quite, in the center of each of the four pieces into which the beam was finally to be broken. Each tag was placed in the bottom of the mould, and held in place with the blade of a shovel until the concrete was poured around and on top of it. The tags were often slightly covered by thin mortar which ran under them, but, being exactly located, they were readily found by scraping the surface.

Placing in Moulds.—The mould for the beam, shown in Fig. 11, was weighed, and the concrete was shoveled into it, and placed and slightly compacted with the aid of large trowels and shovels. The blade of a shovel was generally thrust down by each face of the forms to insure a smooth surface, although this was not found

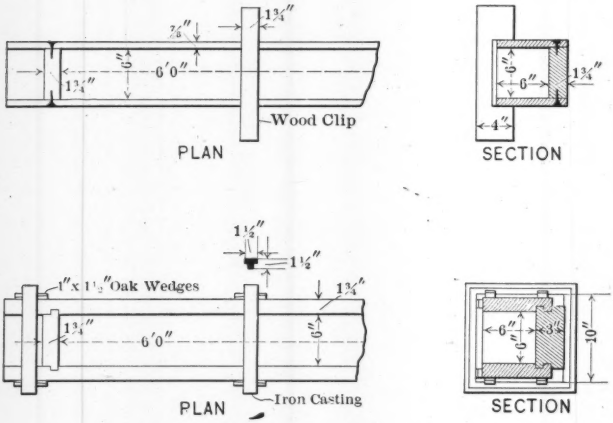


MECHANICAL ANALYSES OF CEMENT, AND JEROME PARK SCREENINGS AND STONE OF 2½-IN MAXIMUM DIAMETER USED IN 1905 BEAM TESTS.

Fig. 10.

absolutely necessary. The mould with the concrete was weighed to obtain the weight of concrete green for the density determination, and placed on one side to set.

The quantity of material required had been calculated approximately from the volumetric tests. If any was left over, after the mould had been filled, it was weighed, and the proportion of this which was water was estimated. As the proportional amount of water may exert a difference of 2 or 3% upon the calculated density of the beam, it is suggested that in future tests the concrete left over be carefully separated from the excess of water, so as to



SKETCH OF MOULD USED FOR MAKING CONCRETE BEAMS.

FIG. 11.

have about the same consistency as the beam, and then weighed, while the water remaining, together with the free water from the surface of the concrete in the mould, be weighed separately and recorded. As this water contains some cement in suspension, its approximate specific gravity should be determined by a test, and this specific gravity should be used in the calculations of the density of the beams.

As soon as the concrete was hard enough to handle easily—usually in about 7 days—it was buried in moist sand, where it remained until it was tested.

TABLE 6.—FORM FOR RECORDING BEAM MIXTURE.

No. of Beam	*217*
Approx. Vol. of Beam	*1¼ cu. ft.*
Date	*May 21, 1905.*
Hour	*1.30* P. M.
Temp. air in cellar	*64°*
CEMENT.	
Kind	*Giant.*
Lot No.	*20.*
Wt., lbs.	*23.32*
AGGREGATE.	
Kind	*Cowe Bay.*
Analysis No.	*Ideal 36% ord.*
Wt., lbs.	*196.68*
WATER.	
Temp. F.	*54°*
Total used	*14.0*
Wt. of Form No. 14 empty	*131.25*
" " " " filled	*364.00*
" " Beam 217	*232.75*
Total Wt. mixed	*234.00*
Wt. of Mix left over	*9.00*
Total Wt. of Beam	*232.75*
Wt. unaccounted for	*1.25*
SIDES OF FORM REMOVED.	
Wt. of Form No. 14 filled	*359.75*
Beams buried—day, hour, days before burying	*3/26/04*
Loss of wt. in setting	*7*

Inspector, *W. H.*

Recording and Computing Data on Beams.—The method of recording the data for mixing is given in Table 6. The form is filled out, in italics, with a typical test, for illustration. When ready to break, after removing from the moist sand, measurements were taken for the purpose of calculating the density and the modulus of rupture.

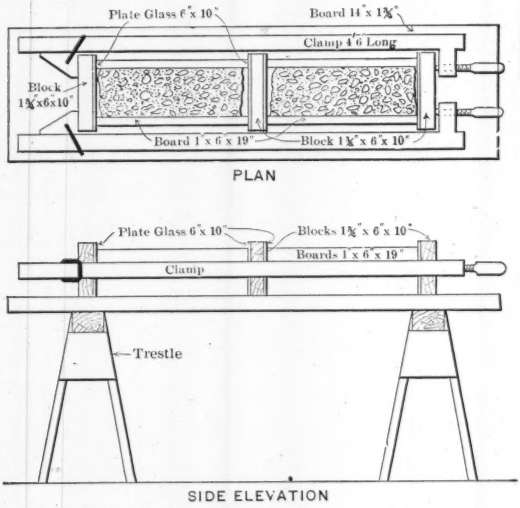
As each beam was broken into four pieces, first in the middle, and then the two halves broken again, measurements were taken at the half and quarter points, also at the two ends, making five sections in all. At each section, three dimensions of depth and three of width were measured, each reading to hundredths of inches. From the beam mixing data and the calculated volume of the beam, the weights of the material in 1 cu. ft. of the beam and the absolute unit volumes, were calculated. The modulus of rupture for each break was calculated from the dimensions and the breaking weight.

Beam Testing Machine.—The machine for breaking the beams was made by Riehlé Brothers, of Philadelphia, and the method of loading it was improved in the laboratory. Fig. 1, Plate XI, is a photograph of the machine with a beam in place ready to break. In order to avoid the negative moment due to overhanging portions of the beam, the bearings for each specimen were 2 in. from each end of the beam, thus giving a variable length between supports for specimens of different length, but an overhang so short as to be negligible in the calculations. Instead of using the poise for weigh-

ing the load upon the beam, an attachment was designed, as shown in the top of the photograph, so that the beam was loaded by dropping shot from a tin funnel into a vessel suspended from the scale beam. This avoided the irregularities incident to the machine as furnished by the manufacturers.

Compression Tests.

As there is no constant relation between the transverse modulus of rupture of concrete and the compressive strength of the same



SKETCH SHOWING METHOD OF CAPPING TEST PRISMS FOR COMPRESSION.

Fig. 12.

mixture, a scheme was devised for obtaining the compressive strength of all the mixtures, in addition to the transverse modulus of rupture. After breaking the beams into four pieces, the two end pieces were capped with neat cement so as to form prisms about 6 in. square and 19 in. long. The method of capping these prisms is illustrated in Fig. 12. Two pieces were capped by the same operation. A piece of smooth, planed, 2-in. plank was laid upon horses,

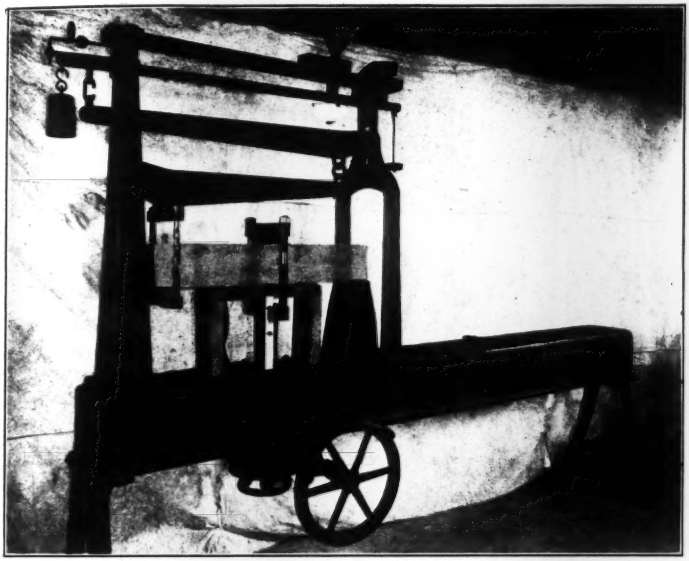


FIG. 1.—MACHINE FOR BREAKING BEAMS.

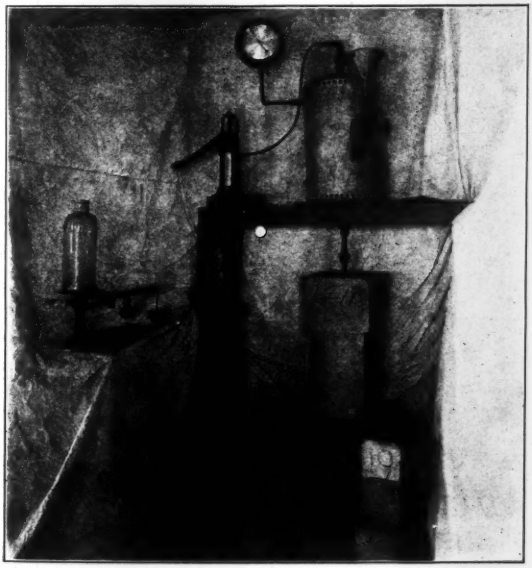


FIG. 2.—APPARATUS FOR TESTING PERMEABILITY OF CONCRETE.

and upright upon this, in a wooden frame, were set four pieces of ¼-in. plate glass, each 6 by 10 in. The lengths of the prisms were gauged by boards, 1 by 6 by 19 in., placed lengthwise and 6 in. apart, so that the broken piece of the beam fitted between them with a space at each end between the rough ends of the beam and the plate glass. The pieces of beams were thoroughly soaked with water before beginning the operation. A plastic paste of neat cement, as stiff as could be readily handled and moulded, was next worked into the spaces between the ends of the specimen and the plate glass, and allowed to set over night. When removed from the mould, each prism was found capped with neat cement having a smooth, glossy surface, and the two ends were parallel. These prisms were sent to the Stevens Institute of Technology, at Hoboken, N. J., and broken in the compression machine. Certain ones were tested there for elasticity, also.

Comparative Comprehensive Strength of True Prisms *versus* Capped Pieces of Beams.

The question naturally arose as to whether the strength of prisms capped with neat cement in this way would correspond to the true compressive strength of the concrete. The neat cement capping, which was of different thickness in the different specimens, might affect the strength, and the specimens might be strained from the rupture in the transverse tests. In order to compare the strength of the capped pieces with the true prisms, one of the beam mixtures, consisting of 1 : 9 concrete with graded Jerome Park 2½-in. aggregate, was used for making up four prisms, 6 by 6 by 18 in., and these true prisms were broken at the same age as the capped pieces of beams mixed with the same ingredients and in like proportions. The results of this comparison are shown in Table 7. It is noticeable that the capped prisms gave a higher average strength than the true prisms, and that there is greater variation between the different specimens. The variation, however, is not great enough to affect the value of the results from the capped pieces, at least, when used in connection with the values of transverse strength. The full results of the compressive tests are given in Table 8.

TABLE 7.

True Prisms.			Capped Pieces of Beams.*		
Reference Numbers.	Compressive strength, in pounds per square inch.	Variations from mean.	Reference Numbers.	Compressive strength, in pounds per square inch.	Variations from mean.
161	1 375	93	155	1 580	153
161	1 240	42	155	1 450	23
161	1 235	47	155	1 250	177
162	1 330	48	157	1 490	63
162	1 315	33	157	1 485	58
162	1 195	87	157	1 305	122
Average.....	1 282	58	Average.....	1 427	99
Percentage...	.....	4.5	.....	.....	14.4

* The compression tests in the other tables are made upon the capped pieces of beams.

Results of Tests.

Table 8 contains a list of the 1905 beam tests, giving the composition, density, transverse strength, and compressive strength of the various mixtures.

Inspection of the quantity of water in the beams, Columns 13 and 20, shows that the smaller stone, that is, the $\frac{1}{2}$-in., requires the most water in gauging, and the coarsest stone, the $2\frac{1}{4}$-in., the least. The Cowe Bay gravel and sand require less water than the broken stone and screenings.

The total volume of voids per cubic foot, Column 24, gives merely the complements of the densities in Column 19. The voids in the cement, Column 22, are based upon the voids in the two neat beams given first in the table, it being assumed in the concrete specimens that the voids due to the cement are in the same ratio to the weight of the cement per cubic foot of concrete as the voids in an average neat beam are to the weight of cement in this neat beam.

In determining the average strength of each mixture, Column 29, all the breaks of the two beams, which are in duplicate, were averaged, that is, if one of a pair of beams had only two breaks, while the other had three, the five breaking strengths were added together, and divided by five, instead of assuming that the two breaks gave the average of one beam and the three breaks the average of the other. The method followed is considered more accurate than

TABLE 8.—COMPLETE LIST OF 1905 BEAM TESTS, GIVING COMPOSITION, TRANSVERSE STRENGTH OF BEAMS, AND COMPRESSIVE STRENGTH OF PRISMS, OF CONCRETE MADE WITH DIFFERENT GRADED AGGREGATES, AND DIFFERENT PROPORTIONS OF GIANT PORTLAND CEMENT.

Beams, 6 by 6 by 72 in. Spans, 68 and 32 in. Prisms, 6 by 6 by 18 in.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)	(27)	(28)	(29)	(30)	(31)	(32)	(33)	(34)	(35)	(36)	(37)
										Weight, in Pounds, of Materials in 1 Cu. Ft. of Beam as Mixed.							Calculated Volume, in Cubic Feet, of Material in 1 Cu. Ft. of Beam as Mixed.					Volume of Voids in 1 Cu. Ft.			Transverse Strength.					Compressive Strength.						
														Totals.													Pounds per square inch.					Pounds per square inch.				
Reference numbers.	Proportions, by weight.	Number of equation of mechanical analysis curve. See Table 10.	Percentage of cement to total dry materials.	Kind of stone.	Kind of sand.	Maximum size of stone, in inches.	Uniformly graded above diameter, in inches.	Percentage finer than diameter in Column 8.	Curve below diameter in Column 8.	Cement.	Aggregate.	Total dry mixed.	Water.	As mixed.	Minimum.	Maximum.	Cement.	Aggregate.	Total dry.	Water.	Total.	Cement.	Aggregate.	Total.	Age, in days.	Number of breaks.	Maximum.	Minimum.	Average.	Age.	Number of breaks.	Maximum.	Minimum.	Average.	Reference numbers.	Remarks.
210	Neat.	.....	.....	.....	.....	.....	.....	.....	.....	104.5	00.0	104.5	29.6	134.1	112.9	133.1	0.541	0.000	0.541	0.475	1.016	0.459	0.000	0.459	89	3	1 177	929	919	.....	.....	.....	.....	.....	210	.....
199	"	.....	.....	.....	.....	.....	.....	.....	.....	100.5	00.0	100.5	30.4	130.9	108.5	130.4	0.520	0.000	0.520	0.487	1.007	0.480	0.000	0.480	91	3	768	744		.....	.....	.....	.....	.....	199	.....
144	1:9	(7)	10	J. P.	J. P.	2½	0.10	29.9	Parabola.	15.0	135.0	150.0	10.2	160.2	151.2	158.9	0.078	0.780	0.858	0.162	1.021	0.066	0.076	0.142	90	3	308	248	276	.....	.....	.....	.....	.....	144	Surface rocky.
146	1:9	(8)	10	"	"	2½	0.10	31.4	"	15.4	138.2	153.6	9.3	162.9	154.8	166.1	0.080	0.799	0.879	0.149	1.028	0.067	0.064	0.131	90	3	266	231	251	.....	.....	.....	.....	.....	146	Excess of fine sand.
148	1:9	(2)	10	"	"	2½	0.20	35.8	Ellipse.	15.0	135.0	150.0	10.2	160.2	151.2	158.9	0.078	0.780	0.858	0.163	1.021	0.066	0.076	0.142	90	3	282	246	265	154	1	1 310	1 310	1 310	148	Surface rocky.
149	1:9	(5)	10	"	"	2½	0.20	36.9	"	14.9	133.7	148.6	9.9	158.5	149.8	158.0	0.077	0.772	0.849	0.159	1.008	0.065	0.086	0.151	91	2	278	253	269	.....	.....	.....	.....	.....	149	Very rocky.
155	1:9	(1)	10	"	"	2½	0.20	37.3	"	15.0	134.5	149.5	10.8	160.3	150.6	158.5	0.078	0.777	0.855	0.173	1.028	0.066	0.079	0.145	90	3	284	244	248	142	3	1 560	1 250	1 425	155	Surface rocky.
157	1:9	(1)	10	"	"	2½	0.20	37.8	"	14.9	134.0	148.9	10.0	158.9	150.1	158.1	0.077	0.775	0.852	0.160	1.012	0.065	0.083	0.148	90	3	258	199		142	3	1 490	1 305	1 425	157	"
150	1:9	(6)	10	"	"	2½	0.20	38.6	"	14.8	133.3	148.1	8.9	157.0	149.3	157.6	0.076	0.771	0.847	0.143	0.990	0.064	0.089	0.153	91	2	293	246	269	149	1	1 405	1 405	1 405	150	"
156	1:9	(3)	10	"	"	2½	0.20	39.4	"	14.6	131.2	145.8	9.6	155.4	147.0	156.3	0.075	0.757	0.832	0.153	0.985	0.064	0.104	0.168	90	3	282	248	247	142	2	1 325	1 225	1 280	156	"
158	1:9	(3)	10	"	"	2½	0.20	39.4	"	14.5	131.0	145.5	9.8	155.3	146.7	156.0	0.075	0.757	0.832	0.157	0.989	0.064	0.104	0.168	90	3	254	201		141	2	1 225	1 185	1 205	158	"
145	1:9	(7)	10	"	"	1	0.10	34.2	Parabola.	14.3	129.0	143.3	11.8	155.1	144.4	155.8	0.074	0.723	0.799	0.189	0.988	0.063	0.138	0.201	90	2	286	241	264	.....	.....	.....	.....	.....	145	.....
147	1:9	(8)	10	"	"	1	0.10	35.0	"	14.4	130.1	144.5	9.9	154.4	145.7	155.2	0.075	0.753	0.828	0.159	0.987	0.064	0.108	0.172	90	2	236	232	234	.....	.....	.....	.....	.....	147	.....
151	1:9	(1)	10	"	"	1	0.10	35.9	Ellipse.	14.3	129.0	143.3	11.9	155.2	144.4	154.5	0.075	0.745	0.820	0.191	1.011	0.064	0.116	0.180	90	3	230	209	221	147	2	1 155	1 015	1 085	151	.....
152	1:9	(1)	10	"	"	1	0.10	35.9	"	14.3	129.0	143.3	11.9	155.2	144.4	154.5	0.075	0.745	0.820	0.191	1.011	0.064	0.116	0.180	90	3	240	203		147	2	1 060	980	1 020	152	.....
159	1:9	(3)	10	"	"	1	0.10	37.7	"	13.9	124.8	138.7	12.8	151.5	139.8	151.6	0.072	0.720	0.792	0.205	0.997	0.061	0.147	0.208	90	2	222	198	212	141	2	930	795	855	159	Surface rocky.
160	1:9	(3)	10	"	"	1	0.10	37.7	"	14.0	125.5	139.5	13.8	153.3	140.6	152.1	0.073	0.725	0.798	0.221	1.019	0.062	0.140	0.202	90	2	223	202		141	2	895	785	840	160	"
164	1:9	(1)	10	"	"	½	0.075	36.3	"	13.4	120.8	134.2	12.9	147.1	135.3	148.7	0.069	0.698	0.767	0.207	0.974	0.059	0.174	0.233	91	2	174	172	200	137	2	845	770	800	164	"
165	1:9	(1)	10	"	"	½	0.075	36.3	"	13.5	121.2	134.7	13.0	147.7	135.8	149.0	0.070	0.700	0.770	0.208	0.978	0.059	0.171	0.230	91	3	232	198		137	2	965	955	960	165	"
153	1:9	(2)	10	"	"	½	0.075	38.2	"	13.3	120.2	133.5	12.8	146.3	134.6	148.2	0.069	0.695	0.764	0.205	0.969	0.059	0.177	0.236	90	3	273	175	208	139	2	895	850	870	153	.....
154	1:9	(2)	10	"	"	½	0.075	38.2	"	13.7	123.0	136.7	13.2	149.9	137.8	150.4	0.071	0.710	0.781	0.212	0.993	0.060	0.159	0.219	90	3	212	197		142	2	890	870	880	154	.....
195	1:9	(3)	10	"	"	½	0.075	39.8	"	13.3	119.5	132.8	14.6	147.4	133.9	147.8	0.069	0.690	0.759	0.234	0.993	0.058	0.183	0.241	90	3	258	218	227	135	2	1 025	1 005	1 015	195	.....
196	1:9	(3)	10	"	"	½	0.075	39.8	"	13.3	119.5	132.8	14.4	147.2	133.9	147.8	0.069	0.690	0.759	0.231	0.990	0.058	0.183	0.241	90	2	205	205		135	2	1 015	910	965	196	.....
168	1: 2⅝: 6⅝	.....	10	"	"	2½	.....	.....	.....	14.6	131.3	145.9	9.6	155.5	147.1	156.3	0.076	0.758	0.834	0.154	0.988	0.064	0.102	0.166	90	3	207	76	265	135	2	980	785	860	168	.....
169	1: 2⅝: 6⅝	.....	10	"	"	2½	.....	.....	.....	14.4	130.0	144.4	10.7	155.1	145.6	155.3	0.075	0.751	0.826	0.172	0.998	0.064	0.110	0.174	90	3	225	201		135	2	920	900	910	169	.....
166	1: 3: 6	.....	10	"	"	2½	.....	.....	.....	14.2	127.7	141.9	8.9	150.8	143.0	153.7	0.074	0.737	0.811	0.143	0.954	0.063	0.126	0.189	91	3	311	225	247	137	2	1 210	1 060	1 135	166	.....
167	1: 3: 6	.....	10	"	"	2½	.....	.....	.....	14.2	127.8	142.0	9.0	151.0	143.1	153.2	0.074	0.738	0.812	0.144	0.956	0.063	0.117	0.180	90	1	224	224		136	2	1 065	965	1 015	167	.....
172	1: 2⅝: 6⅝	.....	10	"	"	1	.....	.....	.....	14.2	127.8	140.0	14.6	156.6	141.1	151.5	0.074	0.736	0.810	0.232	1.042	0.063	0.121	0.184	90	2	168	138	152	148	2	875	855	865	172	.....
173	1: 2⅝: 6⅝	.....	10	"	"	1	.....	.....	.....	14.1	127.2	141.3	11.2	152.5	142.4	153.3	0.073	0.735	0.808	0.180	0.988	0.062	0.130	0.192	91	2	187	117		145	2	965	740	855	173	.....
170	1: 3: 6	.....	10	"	"	1	.....	.....	.....	14.0	126.3	140.3	14.0	154.3	141.4	152.6	0.072	0.730	0.802	0.226	1.028	0.061	0.137	0.198	90	3	205	116	175	133	2	920	860	890	170	.....
171	1: 3: 6	.....	10	"	"	1	.....	.....	.....	13.8	125.4	139.2	12.5	151.7	140.3	151.9	0.071	0.725	0.796	0.200	0.996	0.060	0.144	0.204	90	3	199	160		133	2	855	845	850	171	.....
174	1: 3⅝: 5⅝	....	10	"	"	1	.....	.....	.....	13.7	123.4	137.1	13.6	150.7	138.2	150.6	0.071	0.713	0.784	0.218	1.002	0.060	0.156	0.216	91	3	192	166	186	145	2	890	795	845	174	.....
175	1: 3⅝: 5½	.....	10	"	"	1	.....	.....	.....	13.8	124.0	137.8	13.2	151.0	138.9	151.2	0.071	0.715	0.786	0.211	0.997	0.060	0.154	0.214	91	3	227	171		145	2	980	875	930	175	.....
192	1: 2⅝: 6⅝	.....	10	"	"	½	.....	.....	.....	13.5	121.9	135.4	14.0	149.4	136.5	149.6	0.070	0.703	0.773	0.224	0.997	0.059	0.168	0.227	90	3	211	191	204	137	2	1 105	875	990	192	Slightly rocky.
176	1: 3: 6	.....	10	"	"	½	.....	.....	.....	13.5	121.6	135.1	12.4	147.5	136.2	149.3	0.070	0.702	0.772	0.199	0.971	0.059	0.169	0.228	88	2	135	100	150	143	2	690	670	680	176	.....
177	1: 3: 6	.....	10	"	"	½	.....	.....	.....	13.5	121.0	134.5	13.5	148.0	135.6	148.9	0.070	0.700	0.770	0.216	0.986	0.059	0.171	0.230	88	3	175	169		143	2	835	740	790	177	.....
194	1: 3⅝: 5⅝	.....	10	"	"	½	.....	.....	.....	13.3	119.3	132.6	14.4	147.0	133.7	147.7	0.069	0.689	0.758	0.231	0.989	0.059	0.183	0.242	90	3	156	145	152	135	2	890	815	825	194	.....
187	1:9	(1a)	10	"	"	2½	0.20	37.8	Ellipse.	14.9	134.0	148.9	9.6	158.5	150.1	158.1	0.077	0.775	0.852	0.154	1.006	0.065	0.083	0.148	90	2	269	244	257	141	2	1 535	1 165	1 350	187	Slightly rocky.
188	1:9	(3a)	10	"	"	1	0.10	37.7	"	13.9	125.6	139.5	12.8	152.3	143.6	152.2	0.072	0.725	0.797	0.205	1.002	0.061	0.142	0.203	90	2	230	228	229	141	2	970	915	950	188	.....
198	1:9	(3a)	10	"	"	½	0.075	39.8	"	13.4	121.0	134.4	14.3	148.7	135.5	148.8	0.069	0.700	0.769	0.229	0.998	0.059	0.172	0.231	91	2	181	179	180	145	2	900	875	890	198	.....
178	1:7	(2)	12½	"	"	2½	0.20	35.8	"	18.5	129.6	148.1	9.1	157.2	149.6	157.8	0.096	0.748	0.844	0.146	0.990	0.061	0.075	0.156	88	3	268	208	245	....	2	1 410	1 250	1 330	178	Surface rocky.
179	1:7	(1)	12½	"	"	2½	0.20	37.8	"	18.7	130.8	149.5	11.8	161.3	151.0	158.7	0.097	0.755	0.852	0.189	1.041	0.062	0.066	0.148	92	2	293	135	223	....	1	1 710	1 710	1 710	179	"
180	1:7	(1)	12½	"	"	2½	0.20	37.8	"	18.1	127.5	145.6	11.3	156.9	147.0	156.1	0.094	0.737	0.831	0.181	1.012	0.079	0.090	0.169	92	3	235	223		....	2	1 180	1 090	1 135	180	"
181	1:7	(3)	12½	"	"	2½	0.20	39.4	"	18.4	128.8	147.2	9.7	156.9	148.7	157.3	0.095	0.748	0.838	0.155	0.993	0.061	0.081	0.162	91	3	334	282	272	....	2	1 495	1 410	1 455	181	.....
182	1:7	(3)	12½	"	"	2½	0.20	39.4	"	18.1	127.0	145.1	10.8	155.9	146.5	155.8	0.094	0.735	0.829	0.173	1.002	0.079	0.092	0.171	92	3	292	188		....	2	1 440	1 315	1 375	182	Slightly rocky.
191	1:7	(4)	12½	"	"	2½	0.20	41.3	"	17.5	122.8	140.3	11.2	151.5	141.7	152.8	0.091	0.709	0.800	0.180	0.980	0.077	0.123	0.200	91	2	289	262	276	....	2	1 565	1 450	1 505	191	"

TABLE 8 (*Continued*).—COMPLETE LIST OF 1905 BEAM TESTS, GIVING COMPOSITION, TRANSVERSE STRENGTH OF BEAMS, AND COMPRESSIVE STRENGTH OF PRISMS OF CONCRETE MADE WITH DIFFERENT GRADED AGGREGATES AND DIFFERENT PROPORTIONS OF GIANT PORTLAND CEMENT.

Beams, 6 by 6 by 72 in. Spans, 68 and 32 in. Prisms, 6 by 6 by 18 in.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)	(27)	(28)	(29)	(30)	(31)	(32)	(33)	(34)	(35)	(36)	(37)
										Weight, in Pounds, of Material in 1 Cu. Ft. of Beam as Mixed.							Calculated Volume, in Cubic Feet, of Material in 1 Cu. Ft. of Beam as Mixed.					Volume of Voids, in 1 Cu. Ft.			Transverse Strength.					Compressive Strength.						
														Totals.																						
Reference numbers.	Proportions, by weight.	Number of equation of mechanical analysis curve. See Table 10.	Percentage of cement to total dry materials.	Kind of stone.	Kind of sand.	Maximum size of stone, in inches.	Uniformly graded above diameter, in inches.	Percentage finer than diameter in Column 8.	Curve below diameter in Column 8.	Cement.	Aggregate.	Total dry mix.	Water.	As mixed.	Minimum.	Maximum.	Cement.	Aggregate.	Total dry.	Water.	Total.	Cement.	Aggregate.	Total.	Age, in days.	Number of breaks.	Pounds per square inch. Maximum.	Minimum.	Average.	Age.	Number of breaks.	Pounds per square inch. Maximum.	Minimum.	Average.	Reference numbers.	Remarks.
183	1:5$^{6.7}$	(1)	15	J. P.	J. P.	2½	0.20	37.8	Ellipse.	21.8	123.6	145.4	11.6	157.0	147.1	155.1	0.130	0.715	0.845	0.186	1.081	0.110	0.045	0.155	91	3	361	281	329	....	2	1 920	1 745	1 830	183	.....
184	1:5$^{6.7}$	(1)	15	"	"	2½	0.20	37.8	"	22.0	125.0	147.0	10.9	157.9	148.8	155.6	0.139	0.722	0.861	0.175	1.036	0.118	0.021	0.139	91	2	346	301		....	2	1 715	1 605	1 660	184	Surface rocky.
185	1:5$^{6.7}$	(3)	15	"	"	2½	0.20	39.4	"	21.8	123.4	145.2	11.3	156.5	146.9	156.1	0.113	0.713	0.826	0.181	1.007	0.096	0.078	0.174	90	3	331	300	302	....	2	1 795	1 750	1 770	185	"
186	1:5$^{6.7}$	(3)	15	"	"	2½	0.20	39.4	"	21.7	122.9	144.6	11.4	156.0	146.3	155.0	0.134	0.710	0.844	0.183	1.017	0.105	0.061	0.166	90	3	305	270		....	2	1 370	1 630	1 500	186	.....
190	1:5$^{6.7}$	(4)	15	"	"	2½	0.20	41.3	"	21.8	126.5	148.3	11.5	159.8	150.0	156.0	0.116	0.760	0.876	0.184	1.060	0.098	0.026	0.124	91	3	374	365	369	....	2	1 425	1 395	1 410	190	.....
198	1:11^{5}	(1)	8	"	"	2½	0.20	37.8	"	11.6	133.5	145.1	10.6	155.7	146.0	155.6	0.060	0.771	0.831	0.170	1.001	0.051	0.118	0.169	90	2	221	188	204	....	2	1 100	940	1 020	198	Slightly rocky.
197	1:11^{5}	(3)	8	"	"	2½	0.20	39.4	"	11.6	133.0	144.6	11.4	156.0	145.5	155.4	0.060	0.767	0.827	0.183	1.010	0.051	0.122	0.173	91	3	186	158	171	....	2	965	915	940	197	"
189	1:11^{5}	(4)	8	"	"	2½	0.20	41.3	"	11.5	132.5	144.0	11.8	155.8	144.9	154.9	0.060	0.766	0.826	0.189	1.015	0.051	0.123	0.174	90	3	223	139	184	....	2	970	970	970	189	.....
217	1:8$^{4.3}$	(9)	10.6	Cowe Bay.	Cowe Bay.	2½	0.20	32.1	"	15.2	128.0	143.2	8.3	151.5	144.4	152.2	0.079	0.776	0.855	0.133	0.988	0.067	0.078	0.145	90	3	250	238	230	....	2	1 880	1 555	1 715	217	.....
220	1:8$^{4.3}$	(9)	10.6	"	"	2½	0.20	32.1	"	15.9	133.7	149.6	9.5	159.1	150.9	157.0	0.082	0.810	0.892	0.152	1.044	0.069	0.039	0.108	91	3	244	176		....	2	1 440	1 260	1 350	220	.....
221	1:2$^{2.1}$:5$^{6.2}$	.....	10.6	"	"	2½	.....	.....	.....	14.7	124.0	138.7	9.8	148.5	139.9	149.4	0.076	0.750	0.826	0.157	0.983	0.064	0.110	0.174	91	3	243	211	242	....	2	1 455	1 175	1 315	221	.....
222	1:2$^{2.1}$:5$^{6.2}$	.....	10.6	"	"	2½	.....	.....	.....	14.7	124.0	138.7	9.9	148.6	139.9	149.4	0.076	0.750	0.826	0.159	0.985	0.064	0.110	0.174	91	3	265	228		....	2	1 485	1 310	1 395	222	.....
234	1:8$^{4.3}$	(10)	10.6	"	"	2½	0.20	37.8	Ellipse.	15.3	129.0	144.3	9.2	158.5	145.5	153.0	0.079	0.781	0.860	0.147	1.007	0.067	0.073	0.140	91	3	274	238	256	....	2	1 800	1 800	1 800	234	.....
223	1:8$^{4.3}$	(9)	10.6	"	"	1	0.10	31.2	"	14.9	126.0	140.9	9.8	150.7	142.1	150.9	0.077	0.764	0.841	0.157	0.998	0.065	0.094	0.159	91	3	250	218	258	....	2	1 445	1 145	1 295	223	.....
226	1:8$^{4.3}$	(9)	10.6	"	"	1	0.10	31.2	"	15.1	127.2	142.3	10.4	152.7	143.5	151.8	0.078	0.770	0.848	0.167	1.015	0.066	0.086	0.152	90	3	332	253		146	2	1 565	1 545	1 555	226	.....
227	1:2$^{2.1}$:5$^{6.2}$	.....	10.6	"	"	1	.....	.....	.....	14.8	124.8	139.6	10.8	150.4	140.8	150.1	0.077	0.755	0.832	0.173	1.005	0.065	0.103	0.168	89	3	298	258	248	147	2	2 065	1 660	1 860	227	.....
228	1:2$^{2.1}$:5$^{6.2}$	.....	10.6	"	"	1	.....	.....	.....	15.1	126.3	143.4	9.2	152.6	144.6	152.5	0.078	0.777	0.855	0.147	1.009	0.066	0.079	0.145	89	3	232	214		146	2	1 365	1 185	1 275	228	.....
235	1:8$^{4.3}$	(11)	10.6	"	"	1	0.10	37.7	Ellipse.	14.8	125.0	139.8	9.9	149.7	141.0	150.1	0.078	0.757	0.835	0.158	0.998	0.066	0.099	0.165	90	3	282	262	272	141	2	1 590	1 535	1 580	235	.....
259	1:8$^{4.3}$	(9)	10.6	"	"	½	0.075	33.2	"	14.7	124.0	138.7	10.3	149.0	139.9	150.5	0.076	0.751	0.827	0.165	0.992	0.064	0.109	0.173	91	3	225	199	216	133	2	1 650	1 575	1 610	259	.....
236	1:2$^{2.1}$:5$^{6.2}$	.....	10.6	"	"	½	.....	.....	.....	14.3	120.6	134.9	10.9	145.8	136.0	147.1	0.074	0.730	0.804	0.175	0.979	0.063	0.133	0.196	90	1	170	170	176	141	2	1 265	1 025	1 145	236	.....
237	1:2$^{2.1}$:5$^{6.2}$	.....	10.6	"	"	½	.....	.....	.....	14.7	123.8	138.5	9.6	148.1	139.7	149.4	0.076	0.749	0.825	0.154	0.979	0.064	0.111	0.175	90	1	182	182		141	2	980	895	885	237	.....
224	1:6$^{6.1}$	(9)	13½	"	"	2½	0.20	32.1	Ellipse.	19.3	136.4	145.7	9.3	155.6	147.2	154.1	0.100	0.765	0.865	0.149	1.014	0.084	0.050	0.135	90	3	326	272	323	146	2	2 110	1 670	1 890	224	.....
225	1:5$^{5.0}$	(9)	15.9	"	"	2½	0.20	32.1	"	23.0	121.8	144.8	9.2	154.0	146.6	153.1	0.129	0.738	0.867	0.147	1.014	0.109	0.094	0.133	90	3	352	316		146	2	2 315	1 760	2 040	225	.....
238	1:10$^{7.6}$	(9)	8.5	"	"	2½	0.20	32.1	"	12.4	133.2	145.6	9.6	155.2	146.6	153.6	0.064	0.807	0.871	0.154	1.025	0.054	0.075	0.129	91	2	176	150	168	143	2	1 045	940	990	238	.....
229	1:8$^{4.3}$	(9a)	10.6	"	"	2½	0.20	32.1	"	15.8	133.6	149.4	9.2	158.6	150.7	156.1	0.082	0.810	0.892	0.147	1.039	0.069	0.039	0.108	92	2	288	226	252	143	2	1 415	1 270	1 340	229	.....
230	1:8$^{4.3}$	(9a)	10.6	"	"	2½	0.20	32.1	"	15.7	132.7	148.4	8.6	157.0	149.7	155.6	0.081	0.808	0.884	0.138	1.022	0.069	0.047	0.116	92	3	298	198		143	2	1 790	1 170	1 480	230	.....
231	1:8$^{4.3}$	(9a)	10.6	"	"	1	0.10	31.2	"	15.1	127.0	142.1	9.3	151.4	143.3	151.5	0.078	0.769	0.847	0.149	0.996	0.066	0.087	0.153	92	3	261	240	243	143	2	1 740	1 485	1 610	231	.....
232	1:8$^{4.3}$	(9a)	10.6	"	"	1	0.10	31.2	"	15.2	128.2	143.4	10.1	153.5	144.6	152.3	0.079	0.777	0.856	0.162	1.018	0.067	0.077	0.144	91	1	213	213		142	2	1 205	980	1 090	232	.....
200	1:8$^{8.0}$	(14)	10.2	J. P.	"	2½	0.20	31.2	"	14.6	128.3	142.9	10.2	153.1	144.1	153.7	0.076	0.750	0.896	0.164	0.990	0.064	0.010	0.174	91	3	270	242	256	141	2	1 335	1 230	1 280	200	Surface rocky.
209	1:8$^{8.0}$	(15)	10.2	"	"	2½	0.20	33.0	"	15.3	135.0	150.3	10.0	160.3	151.5	158.5	0.079	0.790	0.869	0.160	1.029	0.067	0.064	0.131	90	3	310	290	298	139	2	1 635	1 605	1 620	209	"
213	1:8$^{8.0}$	(12)	10.2	"	"	2½	0.20	33.8	"	15.4	135.9	151.3	9.5	160.8	152.5	159.2	0.080	0.793	0.873	0.152	1.025	0.068	0.059	0.127	91	3	284	270	289	136	2	2 000	1 800	1 900	213	"
214	1:8$^{8.0}$	(12)	10.2	"	"	2½	0.20	33.8	"	15.4	136.0	145.9	9.9	155.8	147.1	153.7	0.080	0.795	0.875	0.159	1.034	0.068	0.057	0.125	91	3	331	274		136	2	1 925	1 825	1 875	214	"
205	1:2$^{9.2}$:5$^{8.8}$	.....	10.2	"	"	2½	.....	.....	.....	14.7	129.1	143.8	10.1	153.9	144.9	154.3	0.076	0.735	0.831	0.162	0.993	0.064	0.105	0.169	90	1	210	210	172	142	2	1 480	1 120	1 300	205	.....
206	1:2$^{9.2}$:5$^{8.8}$	.....	10.2	"	"	2½	.....	.....	.....	14.8	130.0	144.8	10.5	155.3	146.0	155.0	0.077	0.760	0.837	6.168	1.005	0.065	0.098	0.163	91	1	138	138		140	2	1 435	1 265	1 350	206	.....
219	1:8$^{8.0}$	(16)	10.2	"	"	2½	0.20	37.8	Ellipse.	15.0	132.0	147.0	10.0	157.0	148.2	156.4	0.078	0.772	0.850	0.160	1.010	0.066	0.084	0.150	90	3	295	276	285	135	2	1 775	1 410	1 590	219	.....
201	1:8$^{8.0}$	(13)	10.2	"	"	1	0.10	31.2	"	14.5	128.0	142.5	11.6	154.1	143.7	153.6	0.075	0.748	0.823	0.186	1.009	0.064	0.113	0.177	91	3	348	220	251	141	2	1 375	1 265	1 320	201	Surface rocky.
202	1:8$^{8.0}$	(13)	10.2	"	"	1	0.10	31.2	"	14.5	127.8	142.3	10.8	153.1	143.5	153.5	0.075	0.745	0.820	0.173	0.993	0.064	0.116	0.180	91	3	290	246		142	2	1 565	1 550	1 560	202	"
210	1:8$^{8.0}$	(15)	10.2	"	"	1	0.10	33.2	"	14.5	127.6	142.1	10.3	152.4	143.3	153.3	0.075	0.745	0.820	0.163	0.983	0.064	0.116	0.180	90	3	300	280	288	139	2	1 875	1 660	1 770	210	.....
208	1:2$^{9.3}$:5$^{9.8}$	.....	10.2	"	"	1	.....	.....	.....	14.5	127.5	142.0	9.7	151.7	143.2	153.2	0.075	0.745	0.820	0.156	0.976	0.064	0.116	0.180	90	3	298	234	276	141	2	1 745	1 555	1 650	208	.....
204	1:2$^{9.3}$:5$^{9.8}$	.....	10.2	"	"	1	.....	.....	.....	14.4	126.7	141.1	9.8	150.9	142.3	152.6	0.075	0.740	0.815	0.157	0.972	0.064	0.121	0.185	90	2	298	255		141	2	1 590	1 435	1 475	204	.....
215	1:8$^{8.0}$	(17)	10.2	"	"	1	0.10	37.7	Ellipse.	14.4	126.7	141.1	11.0	152.1	142.3	152.6	0.075	0.740	0.815	0.176	0.991	0.064	0.121	0.185	91	3	261	241	249	136	2	1 545	1 345	1 445	215	.....
211	1:8$^{8.0}$	(15)	10.2	"	"	½	0.075	34.4	"	13.9	122.3	136.2	12.8	149.0	137.3	149.5	0.072	0.715	0.787	0.205	0.992	0.061	0.152	0.213	90	1	196	196	205	139	2	1 175	1 085	1 130	211	.....
212	1:8$^{8.0}$	(15)	10.2	"	"	½	0.075	34.4	"	13.7	120.4	134.1	14.9	149.0	135.2	148.1	0.071	0.705	0.776	0.239	1.015	0.060	0.164	0.224	90	3	217	202		139	2	1 170	1 080	1 125	212	.....
207	1:2$^{9.2}$:5$^{8.8}$	.....	10.2	"	"	½	.....	.....	.....	13.8	121.3	134.1	12.4	147.5	135.2	147.6	0.073	0.710	0.783	0.199	0.982	0.062	0.155	0.217	91	3	222	210	212	140	2	1 385	1 235	1 310	207	.....
208	1:2$^{9.2}$:5$^{8.8}$	.....	10.2	"	"	½	.....	.....	.....	14.0	123.0	137.0	12.3	149.3	138.1	150.0	0.072	0.719	0.791	0.197	0.988	0.061	0.148	0.209	91	3	235	192		140	2	1 205	1 145	1 175	208	.....
218	1:6$^{8.7}$	(12)	12½	"	"	2½	0.20	33.8	Ellipse.	19.2	131.3	150.5	9.4	159.9	152.0	158.9	0.099	0.767	0.866	0.151	1.017	0.084	0.050	0.134	90	3	413	353	379	135	2	2 280	2 150	2 190	218	Slightly rocky.
216	1:5$^{5.4}$	(12)	15.3	"	"	2½	0.20	33.8	"	22.7	126.0	148.7	9.6	158.3	150.5	157.9	0.117	0.735	0.852	0.154	1.006	0.099	0.049	0.148	91	3	445	391	409	136	2	2 440	1 840	2 140	216	.....

the other, because a beam which can be broken only once or twice is possibly imperfect, and therefore this beam should not have quite so large a place in the average as the beam with three breaks.

The compressive strength of the concrete prisms made by capping two pieces of each beam are given at the right of each sheet of Table 8. These prisms were broken by Professor Frederick L. Pryor at the Stevens Institute, and although the heads of the machine were fixed, the ends of the specimen were sufficiently parallel to give good results, nearly all the pieces breaking in a manner normal to long prisms. The breaks, as is usual with such specimens, were more longitudinal than is the case with cubes, where two pyramids are generally formed with their bases against the heads of the machine.

In computing Table 8 the following data have been used:

Volumes, cubic feet per 100 lb. as mixed:

Cement, 1.00;
Jerome Park screenings (crusher run), 1.06;
Jerome Park stone (crusher run), 1.03;
Jerome Park screenings and stone vary with the mixture;
Cowe Bay sand (natural), 1.11;
Cowe Bay gravel, 0.97.

Specific gravities:

Cement, 3.10;
Jerome Park screenings vary with the size;
Jerome Park stone, 2.78;
Mixture of Jerome Park screenings and stone, 2.77;
Cowe Bay sand, 2.65;
Cowe Bay gravel, 2.65.

Weights per cubic foot, as mixed:

Cement, 1.00;
Jerome Park screenings (crusher run), 94.8;
Jerome Park stone, 97.0;
Mixture of Jerome Park screenings and stone varies with the mixture;
Cowe Bay sand (natural), 90.0;
Cowe Bay gravel, 102.7.

Column 16 = Column 11 × 0.08 + Column 13.

Column 17 = Column 13 + (Column 25 × 62.4).

Column 16 represents the theoretical minimum, assuming 8% of water for chemical combination.

Column 17 represents the theoretical maximum, assuming the voids to be filled with water.

Comparative Tests of Density and Strength Forming the Basis of Conclusions.

In order to bring out more clearly the comparative relations of the beams tested which have warranted the conclusions stated at the beginning of this paper, short tables grouping the essential facts which form the basis of the conclusions are presented in the following pages.

Concrete with Aggregates of Different Maximum Size. Conclusions 1 and 2.

In Table 9 are shown the averages of density and of breaking tests of concrete mixtures of cement and aggregates of different maximum size. The figures represent the averages of all specimens tested, according to each character of mixing, with the exception of four specimens omitted, where the exceptionally low strength is due to voids not being filled. In every case but one the mixture with 2½-in. stone for the maximum size is denser than that with 1-in. stone, and in every case but one the 1-in. is denser than the ½-in. The modulus of rupture and the compressive strength of the specimens with the three sizes of stone follow the same general order, the concrete with coarse stone being always stronger than the finer, except in one case where the 1-in. is 1% stronger than the 2½-in.

The general averages are calculated at the bottom of the table, and also the ratios of density and strength, based on the 2½-in. material as unity. Comparing these ratios with the ratios of strength with different percentages of cement, as given in Table 15, it appears that an additional amount of cement to the extent of about one-sixth part with the maximum aggregate 1 in., and one-third part with the maximum aggregate ½ in., will be required to produce a concrete equal in strength to a concrete having a maximum aggregate of 2½ in.

TABLE 9.—AGGREGATES OF DIFFERENT MAXIMUM SIZE. COMPARATIVE DENSITY AND STRENGTH.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
REFERENCE NUMBERS.			Proportion, by weight.	MATERIAL.		Character of mixture.	AVERAGE DENSITY.			AVERAGE MODULUS OF RUPTURE AT 90 DAYS, IN POUNDS PER SQUARE INCH.			AVERAGE COMPRESSIVE STRENGTH AT 140 DAYS, IN POUNDS PER SQUARE INCH.		
2¼-in. stone.	1-in. stone.	½-in. stone.		Stone.	Sand.		2¼-in. stone.	1-in. stone.	½-in. stone.	2¼-in. stone.	1-in. stone.	½-in. stone.	2¼-in. stone.	1-in. stone.	½-in. stone.
144, 146, 148, 149, 150, 155, 156, 157, 158.	145, 147, 151, 152 159, 160.	164, 165, 153, 154, 195, 196.	1:9	J. Park.	J. Park.	Ideal.	0.851	0.810	0.767	257	226	208	1 342	950	915
166, 167, 168, 169.	170, 171, 172, 173, 174, 175.	176, 177, 192, 194.	1:9	"	"	Natural.	0.821	0.798	0.768	211	171	162	980	879	821
187.	188.	198.	1:9	"	"	Uniform aggregate.	0.832	0.797	0.769	257	229	180	1 350	950	890
217, 220, 221, 222, 229, 230, 234.	223, 226, 228, 231, 232, 235.	236, 237, 259.	1:9	Cowe Bay.	Cowe Bay.	Various.	0.859	0.847	0.853	243	246	189	1 486	1 402	1 231
209, 213, 214.	203, 204, 210, 215.	207, 208, 211, 212.	1:9	J. Park.	Cowe Bay.	"	0.872	0.818	0.784	291	273	207	1 798	1 585	1 185
	Averages.....						0.847	0.814	0.788	252	229	189	1 391	1 153	1 008
	Ratios.....						1.00	0.96	0.93	1.00	0.91	0.75	1.00	0.83	0.72

CONCRETE WITH JEROME PARK *vs.* COWE BAY *vs.* MIXED AGGREGATES. Conclusions 3, 4 and 5.

Tables 10 and 11 give the comparative density and strength of concrete made with Jerome Park stone and screenings *vs.* Cowe Bay gravel and sand *vs.* Jerome Park stone and Cowe Bay sand. The proportions given are 1 : 9 and 1 : 3 : 6 by weight. This is not strictly true, because, in order to make a true comparison of the different materials, a correction was made for the Cowe Bay mixtures. Actually, the proportions for the straight Cowe Bay materials are 1 ; 8.43, and 1 : 2.81 : 5.62, and the actual proportions by weight for the mixed materials are 1 : 8.80 and 1 : 2.92 : 5.88. By making this correction, the proportions by absolute volume are exactly the same, and, as it is this which affects the composition, it is the proper method of proportioning in comparative tests. The results, accordingly, show the true relation between the rounded gravel and the broken stone.

The values in Table 10 are the results of tests of concrete composed of the aggregates and 10%, by weight, of cement graded to the best elliptical curve for each material, which represents in general the best possible mixture of each material with the given percentage of cement. In general, the straight Cowe Bay material produces the greatest density. This is undoubtedly due to the rounded character of the grains, and is in accord with the results of other experimenters. The mixed Jerome Park stone and Cowe Bay sand forms a concrete less dense than the straight Cowe Bay material, but denser than the straight Jerome Park; in other words, it is intermediate between the two.

The breaking strength does not follow the same direction as the density. The most noticeable difference is between the straight Cowe Bay and the mixed Jerome Park stone and Cowe Bay sand. The straight Cowe Bay material, although producing a denser mixture, is not so strong as the mixed material in two cases out of three, even although, as shown in Column 18 of Table 8, the amount of cement in a cubic foot of the Cowe Bay concrete is greater than in a like volume of the other. An examination of the tests indicates that the one test with ½-in. stone which does not follow this rule is the erratic one rather than the other two, since in the transverse tests the difference in the ½-in. specimens

TABLE 10.—JEROME PARK *vs.* COWE BAY *vs.* MIXED AGGREGATES, EACH GRADED TO THE BEST ELLIPTICAL CURVE FOR THAT MATERIAL. COMPARATIVE DENSITY AND STRENGTH.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
Reference numbers.	Proportion, by weight.	Maximum size of stone, in inches.	Density.			Modulus of Rupture at 90 Days, in Pounds per Square Inch.			Compressive Strength at 140 Days, in Pounds per Square Inch.		
			J. Park stone and screenings.	C. Bay gravel and sand.	J. Park stone. C. Bay sand.	J. Park stone and screenings.	C. Bay gravel and sand.	J. Park stone. C. Bay sand	J. Park stone and screenings.	C. Bay gravel and sand.	J. Park stone. C. Bay sand.
155, 157; 217, 220; 213, 214....	1 : 9	2½	0.854	0.874	0.874	243	230	289	1 425	1 533	1 888
151, 152; 223, 226; 210.....	1 : 9	1	0.820	0.845	0.820	221	258	288	1 052	1 425	1 770
164, 165; 259; 211, 212.....	1 : 9	½	0.769	0.827	0.787	200	216	212	880	1 610	1 243
Averages.....	.....	.....	0.814	0.849	0.827	221	235	263	1 119	1 523	1 634

NOTE.—Weights of Cowe Bay material are corrected for difference in specific gravity, so as to have the same relative absolute volumes as the Jerome Park material.

See Table 5 for formulas.

is only 2%, whereas the difference in the tests of larger (2½-in. and 1-in.) stone is much greater, while in the compressive tests the strength of the ½-in. specimens, which averages 1 610 lb., appears abnormally large, being, in fact, greater than the tests above it with the coarser stone. It would appear from this that broken stone concrete of mica schist rock is stronger than gravel concrete under like conditions, and that this increase in strength is due to the difference in the surface of the coarse particles of stone. This, in general, is in accordance with the results of the best experiments, notably those of E. Candlot, in France, although it has been disputed in many quarters.

In Table 11 the mixtures of the different materials are in the same proportions (corrected for different specific gravity), and are not those demanded by the best analyses for each material. The results, therefore, are not apt to be so conclusive as those scheduled in Table 10, but it will be seen that they follow in general the same direction. The straight Cowe Bay gravel and sand mixture, while considerably denser than the mixed Jerome Park stone and Cowe Bay sand, is generally of lower strength than the latter. Tables 10 and 11 both indicate that a mixture of Jerome Park stone and screenings gives a concrete of lower density and lower strength than either the straight Cowe Bay mixtures or the mixed Jerome Park stone and Cowe Bay sand. In the 1 : 3 : 6 mixtures, Table 11, this might be due in part to the fact that the materials are not graded to the proportions which are best for them, but this difference is eliminated in Table 10. Further tests are essential to determine whether the principles just stated apply to other classes of broken stone. The age of the specimens also tends to affect the relative strength, because, if the mixture is rich enough, as the concrete becomes older there is more and more tendency in compressive tests for the stones to shear, so that the actual strength of the particles of stone become more and more a function of the strength of the concrete.

The results from these experiments, which indicate that sand with rounded grains produces a stronger concrete than broken-stone screenings in the same proportions, are apparently contradictory to certain tests by other experimenters. These other results may probably be accounted for either by the difference in the relative

TABLE 11.—JEROME PARK *vs.* COWE BAY *vs.* MIXED AGGREGATES, IN PROPORTIONS 1:3:6 BY WEIGHT. COMPARATIVE DENSITY AND STRENGTH.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
Reference numbers.	Proportion, by weight.	Maximum size of stone, in inches.	Density.			Modulus of Rupture at 90 Days, in Pounds per Square Inch.			Compressive Strength at 140 Days, in Pounds per Square Inch.		
			J. Park stone and screenings.	C. Bay gravel and sand.	J. Park stone, C. Bay sand.	J. Park stone and screenings.	C. Bay gravel and sand.	J. Park stone, C. Bay sand.	J. Park stone and screenings.	C. Bay gravel and sand.	J. Park stone, C. Bay sand.
166, 167; 221, 222; 205, 206....	1:3:6	2½	0.812	0.826	0.834	247	242	174*	1 075	1 355	1 325
170, 171; 227, 228; 209, 204....	1:3:6	1	0.799	0.844	0.818	175	248	276	870	1 567	1 562
176, 177; 236, 237; 207, 208....	1:3:6	½	0.771	0.815	0.787	150	176	212	735	1 015	1 242
Average.....	.....	.....	0.794	0.828	0.813	191	222	221	893	1 313	1 376

NOTE.—Weights of Cowe Bay material are corrected for difference in specific gravity, so as to have the same relative absolute volumes as the Jerome Park material.

*Only two breaks.

sizes of the sand and the screenings, or else by the possibility that in some cases the chemical composition of the screenings is such that the fine particles form a chemical combination with the cement.

Concrete with Aggregate Graded by Ideal Mechanical Analysis Curves *vs.* Mixtures of Natural Materials in Ordinary Proportions.

Conclusion 6.

Table 12 compares the density and strength of concrete graded by ideal mechanical analysis curves with the density and strength of natural proportions. The tests of graded mixtures in nearly every case are higher than the natural proportions. Greater differences would have appeared if the best ideal and best natural proportions had been selected instead of averaging all of them.

Instead of using the average of all the specimens made by artificial curves and the natural proportions, if a selection had been made of the specimens proportioned by the best ideal analysis curve with 2½-in. maximum stone—that is, by the lowest artificial curve with which the voids were filled, and by the best natural mixtures with the same size of stone, which, on the whole, were the 1 : 3 : 6 proportions—the average value of the modulus of rupture would have been 241 lb. per sq. in. for the ideal, and 211 lb. per sq. in. for the natural proportions, representing an increase in strength of 14% by using the ideal mixtures. The relative saving in cement is estimated with the aid of the data in Table 15.

Concrete with Aggregates Graded by Ideal Mechanical Analysis Curves Having Graded Coarse Aggregate *vs.* Aggregates Graded Similarly in the Sand Portion, but with Coarse Aggregate of Uniform Size.

Conclusion 7.

Trial mixtures of concrete made in the laboratory with uniform coarse stone gave greater density than could be obtained with uniformly graded stone. However, in the former, nearly 50% of the weight of total aggregate consisted of particles of the coarsest diam-

TABLE 12.—AGGREGATES GRADED BY IDEAL MECHANICAL ANALYSIS CURVES *vs.* MIXTURES OF NATURAL MATERIALS IN ORDINARY PROPORTIONS. COMPARATIVE DENSITY AND STRENGTH.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
Reference number. Ideal mixtures.	Reference number. Natural mixtures.	Proportion, by weight.*	Materials.		Maximum size of stone, in inches.	Average Density.*		Average Modulus of Rupture at 90 Days, in Pounds per Square Inch.†		Average Compressive Strength at 140 Days, in Pounds per Square Inch.†	
			Stone.	Sand.		Ideal.	Natural.	Ideal.	Natural.	Ideal.	Natural.
144–146–148	166–167	1:9	J. Park.	J. Park.	2½	0.851	0.821	257	211	1 342	980
149–150–155	168										
156–157–158	169										
145–147–151	170–171–172	1:9	"	"	1	0.810	0.798	226	171	950	873
152–159–160	173–174–175										
153–154–164	176–177	1:9	"	"	½	0.767	0.768	208	162	915	821
165–195–196	192–194										
217–220	221–222	1:9	Cowe Bay.	Cowe Bay.	2½	0.863	0.826	230	242	1 535	1 355
223–226	227–228	1:9	"	"	1	0 845	0.844	258	249	1 425	1 505
259	236–237	1:9	"	"	½	0.827	0.817	216	176	1 610‡	1 040
200–209	.....	1:9	J. Park.	"	2½	0.861	0.834	283	172‡	1 669	1 325
213–214	205–206										
210	203–204	1:9	"	"	1	0.820	0.818	288	276	1 770	1 565
211–212	207–208	1:9	"	"	½	0.782	0.787	202	212	1 125	1 240
Average....	.....	.....	.....	.....	.....	0.821	0.810	236	212	1 334	1 172

*In Cowe Bay mixtures the proportions are corrected for specific gravity so as to be the same by absolute volume.

†Average of all specimens tested by each character of mixture, except omitting 201, 202, 205 and 206, where an exceptionally low test is evidently due to voids not being filled.

‡Only two breaks.

eter of stone, and, in mixing, the concrete appeared to be very coarse, so that it was considered doubtful whether the concrete would work well in practice because of the apparent tendency of the mortar to run away from the stones, and leave pockets of stone and voids. Accordingly, in this 1905 series of experiments, only a few beams were made with the coarse stone of uniform size, in order to determine whether the results were sufficiently good to warrant the continuation of this line of tests at a future date, if circumstances permitted.

The results as indicated in Table 13 show no marked difference between the specimens with graded coarse stone and those with uniform coarse stone. The former average slightly higher in strength, while the latter have slightly greater density. This is contrary to what would be expected, since the tests in general indicate that the densest mixtures produce the strongest concrete. The fact can be explained at the present time only by assuming that the uniform coarse stone does tend to separate, as suggested above, and thus produces a less homogeneous concrete. Further tests are necessary, however, to determine whether this is true or whether these few results are abnormal. The analysis curves used in the mixtures of uniform stone are indicated in the diagrams. They were adopted as a result of the volumetric or density tests. The density tests indicated that a slightly better mixture would have been obtained by lowering the curve at the juncture of the ellipse and the first straight line, but since they also indicated that this style of curve required more very fine sand than a curve with graded coarse stone, it was thought best to adopt the curves shown, and then use, for comparison with them, the specimens with the graded stone, the analysis curve of which, in the sand portion, was the same ellipse.

Influence of the Analysis of the Coarse Aggregate upon the Strength of the Concrete.—One of the most important conclusions which may be drawn from the comparative tests shown in Table 13 is that there is comparatively little difference in the density and strength of concrete, whatever may be the analysis of the coarse aggregate (meaning by the "coarse aggregate" all the particles the sizes of which are greater than a diameter which is one-tenth the maximum diameter of the stone), provided this coarse aggregate

TABLE 13.—AGGREGATES GRADED BY IDEAL MECHANICAL ANALYSIS CURVES HAVING GRADED COARSE AGGREGATE *vs.* AGGREGATES GRADED SIMILARLY IN THE SAND PORTION, BUT WITH COARSE AGGREGATE OF UNIFORM SIZE. COMPARATIVE DENSITY AND STRENGTH.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
Reference number. Graded stone.	Reference number. Uniform stone.	Proportion, by weight.	Materials.		Maximum size of stone, in inches.	Average Density.		Average Modulus of Rupture at 90 Days, in Pounds per Square Inch.		Average Compressive Strength at 140 Days, in Pounds per Square Inch.	
			Stone.	Sand.		Graded coarse stone.	Uniform coarse stone.	Graded coarse stone.	Uniform coarse stone.	Graded coarse stone.	Uniform coarse stone.
155, 157	187	1:9	J. Park.	J. Park.	2½	0.854	0.832	250	257	1 425	1 350
159, 160	188	1:9	"	"	1	0.795	0.797	212	220	848	950
195, 196	198	1:9	"	"	½	0.759	0.769	219	180	990	890
217, 220	229, 230	1:9	Cowe Bay	Cowe Bay.	2½	0.853	0.888	231	252	1 535	1 411
223, 226	231, 232	1:9	"	"	1	0.845	0.852	258	233	1 425	1 352
Average.	.....	.....	.....	.....	.....	0.821	0.828	234	230	1 245	1 191

In Cowe Bay mixtures the proportions are corrected for specific gravity so as to be the same by absolute volume.

does not contain a greater weight of medium-sized particles than can be represented by a straight line. In other words, the coarse aggregate may be uniformly graded above the size corresponding to one-tenth the diameter of the coarsest particles, or it may contain an excess of uniform-sized stone the diameter of which is the same as the coarsest diameter of the graded stone. Expressed in another way, the effect upon the density and the strength is practically the same whether uniform stone or uniformly graded stone, or a mixture between these two, is used. However, tests should be made on a large scale, with concrete mixed by hand and by machine, before adopting uniform stone in regular construction work, in order to prove conclusively that the stones in the mixture of uniform stone do not separate from the mortar; in other words, that the concrete does not work harshly.

While the tests of strength do not include tests to determine whether or not it might be possible to use an excess of medium-sized stone over and above what would be contained in a uniformly-graded coarse aggregate, the tests of density indicate that when the curve of the coarse aggregate is raised above a straight line, the density is immediately decreased, signifying that the uniformly-graded coarse aggregate or an aggregate having a curve which is below a straight line, is the best to use.

Effect of Fineness of Sand upon Density and Strength.

Conclusions 8 and 9.

The relative effect of sand and stone upon the density and strength is evident from the preceding paragraphs. The density of the concrete is affected very much more by the variation in diameter of the sand particles than of the stone particles, an excess of fine or medium sand decreases both the density and strength of the concrete. The effect of the fineness of the sand, that is, the proportions of the different-sized grains, was carefully studied in the volumetric tests which have been described, and the results were confirmed by laboratory and field tests.

Relation of Ideal Curves of Different-Sized Stone.

From the preceding paragraphs it is evident that the equations of curves given on page 90 for aggregate plus cement show that, for

a given character of aggregate, a single equation in which the only variable is the diameter of the particles of maximum size will apply to aggregates having a maximum diameter varying from $\frac{1}{2}$ in. to $2\frac{1}{4}$ in.

Effect of Density and Strength of Concrete by Increasing the Percentage of Sand and Decreasing the Percentage of Stone.

Conclusion 12.

The density experiments indicated very distinctly that, the best ideal curve of mechanical analysis having been found, raising this curve in any portion of its length decreases the density of the mixture. Accordingly, to test this point in the beam tests, as shown in the diagrams, Figs. 5 to 10, specimens were also made with each material proportioned according to a curve which was raised above the best ideal at the tangent point of the ellipse and straight line, the lower portion of these higher curves being still in the form of an ellipse with the same horizontal or major axis but with a longer minor axis. More simply expressed, this is equivalent to increasing the sand and decreasing the stone. It is evident from an examination of the full data in Table 8 that the results of this comparison were not as conclusive as might be desired. This is partly due to the slight difference between the various curves, to the comparatively few tests made with each curve, to the variation which must be expected in all tests of concrete, and to the fact that in some of the lowest curves the voids were not entirely filled and therefore the results are less accurate. With the natural mixtures, the same rather inconclusive results were obtained, the proportions selected being so nearly alike that the difference is not marked. The general trend of the tests, however, indicates that the theory assumed is correct, and it is certainly substantiated by previous tests, made by one of the writers at Little Falls, which indicate quite conclusively that, with the same proportions of cement to total aggregate, the strongest mixture is that with the largest possible proportion of stone and the smallest possible proportion of sand.

In Table 14 are given the results selected from these tests at Little Falls, N. J., made in 1901. All the specimens presented

have the same proportions by weight, namely 1 : 8. The ratio of sand to stone, however, is varied from 2 : 6 to 8 : 0. It is seen from an inspection of the table that in every case there is an increase in both density and strength as stone is substituted for an equal weight of sand.

TABLE 14.—EFFECT OF INCREASING THE PERCENTAGE OF SAND AND DECREASING THE STONE, FROM THE LITTLE FALLS EXPERIMENTS.

Proportion, by weight, of cement to total aggregate.		Absolute volume of cement.	Density, *i. e.* total cement + sand + stone.	Average modulus of rupture at 33 days, in pounds per square inch.
1:8	1:2:6	0.091	0.865	319
1:8	1:3:5	0.086	0.833	285
1:8	1:4:4	0.082	0.801	209
1:8	1:5:3	0.080	0.799	151
1:8	1:6:2	0.076	0.760	102
1:8	1:8:0	0.073	0.754	41

PROPORTIONING SAND AND STONE IN PRACTICE.

Conclusion 12.

Where two aggregates are used, such as natural sand and natural gravel, or crusher-run screenings and crusher-run broken stone, it is evident from the preceding paragraphs that the relative proportions of sand to stone may be properly graded in the field from day to day by using in all cases as little sand as possible, while avoiding visible voids in the concrete. In practice, when proportioned in this way, the mixture is found to be not only theoretically denser and stronger, but it works most smoothly in placing. Having selected a ratio of cement to total aggregate, this gives a better and easier method of proportioning in cases where it is not considered economical to use more than two aggregates—that is, where artificial grading is prohibited—than any number of theoretical tests. This plan is also especially advantageous where the sand varies in fineness from day to day; for, the finer the sand, the farther it will spread, and the less of it is required to fill the voids of the stone, and also the less of it should be used in proportion to the cement, since a mortar of fine sand is weaker than a mortar of coarse sand.

Concrete with Different Percentages of Cement.

Conclusion 10.

Table 15 shows the density and strength of beams in which different percentages of cement are used to the weight of the total dry material, ranging from 8 to 15%, and in proportions by weight from 1 : 11.5 to 1 : 5.7, thus covering all ordinary proportions in practice. As the percentage of cement is increased, the strength increases, and in nearly similar ratios.

Effect upon Density of Substituting Cement for Fine Sand.

Conclusions 10 and 11.

The average densities in Table 15 show an extreme variation of less than 1 per cent. This is in confirmation of the statement already made that a substitution of cement for fine sand having grains of the same diameter does not affect the density. This is also proved still more definitely in the volumetric tests of density. As the mechanical analysis curves in all the beam mixtures include both cement and aggregate, in the specimens with different percentages of cement the additional cement is actually substituted for sand having grains of the same size.

Permeability Tests.

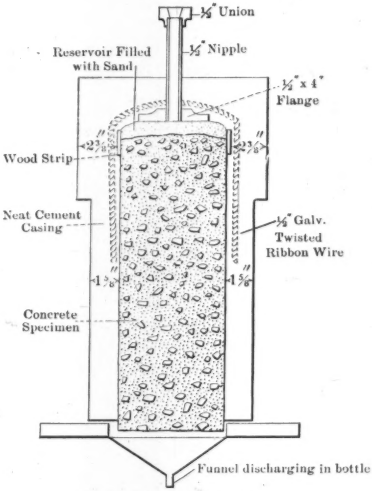
The results of the permeability tests have emphasized the fact that little is known of the action of concrete in resisting the flow of water. Examination of the tests, which are given in full, in Table 18, indicates in general that, using different proportions and different sizes of the same class of materials, the laws of water-tightness are somewhat similar to those of strength. If the percentages of cement be the same, the specimens having the greatest density are usually most water-tight, and in the specimens having similar density but different percentages of cement, the water-tightness increases with the percentages of cement. The ratios, however, are very different from the ratios of either density or strength, a slight difference in the composition producing a great effect upon the water-tightness. Different kinds of aggregates, also, for reasons not yet explained, produce very different results in water-tightness. The results of the tests are discussed in detail

TABLE 15.—COMPARATIVE DENSITY AND STRENGTH OF SIMILAR CONCRETE WITH DIFFERENT PERCENTAGES OF CEMENT AND 2¼-INCH STONE GRADED AS AN ELLIPSE AND STRAIGHT LINE.

(1)	(2)		(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
Reference numbers.	Materials.		Uniformly graded above diameter, in.	Percentage finer than diameter in Column 3.	Density with different percentages of cement.*				Modulus of rupture at 90 days, different percentages of cement.*				Compressive strength at 140 days, different percentages of cement.			
	Stone.	Sand.			8%	10%	12½%	15%	8%	10%	12½%	15%	8%	10%	12½%	15%
193, 197, 198	J. Park.	J. Park.	0.20	37.8-39.4	0.829	.....	.....	.....	188	.....	.....	.....	980	.....	.....	.....
148, 155, 156	"	"	0.20	35.8, 37.8, 39.4	.....	0.846	.....	.....	.....	250	.....	.....	.....	1 129	.....	.....
157, 158	"	"	0.20	37.8-39.4												
178, 179, 180	"	"	0.20	35.8, 37.8, 37.8	.....	.....	0.832	.....	.....	.....	245	.....	.....	.....	1 418	.....
181, 182, 191	"	"	0.20	39.4, 39.4, 41.3												
183, 184	"	"	0.20	37.8, 37.8	.....	.....	.....	0.839	.....	.....	.....	326	.....	.....	.....	1 634
185, 186, 190	"	"	0.20	39.4, 39.4, 41.3												
233	C. Bay.	C. Bay.	0.20	32.1	0.871	.....	.....	.....	163	.....	.....	.....	990	.....	.....	.....
217	"	"	0.20	32.1	.....	0.855	.....	.....	.....	245	.....	.....	.....	1 715	.....	.....
224	"	"	0.20	32.1	.....	.....	0.865	.....	.....	.....	307	.....	.....	.....	1 890	.....
225	"	"	0.20	32.1	.....	.....	.....	0.867	.....	.....	.....	339	.....	.....	.....	2 040
Averages..	.....	.....	.....	.....	0.850	0.850	0.848	0.853	176	248	276	332	985	1 428	1 654	1 837

*In Cowe Bay mixtures the percentage of cement by weight is increased in each case to balance the difference in specific gravity between this and the Jerome Park material.

on pages 132 to 137. An important result of the permeability tests has been the evolution of an apparatus, described in succeeding paragraphs, by which almost any character, shape or thickness of specimen may be submitted to water pressure. The advantages of the apparatus lie in the exposure of the entire top surface of the specimen to the water pressure, the coating of the sides of the specimen with neat cement so as to confine the flow, and the discharge of the water through the bottom of the specimen.



METHOD OF PREPARING SPECIMEN FOR PERMEABILITY TESTS.

FIG. 13.

Method of Making Permeability Tests.—The apparatus used in making permeability tests and the preparation of the specimen are indicated in Fig. 13, and in Fig. 2, Plate XI. The method of preparing the specimen which was found satisfactory after a number of trials is illustrated in Fig. 12. A piece of a concrete beam, about 6 in. square and 17 in. long, obtained by breaking the beam in the regular manner in the testing machine, is scored over the surface of its four sides by a hammer and cold chisel, so as to

give rough and uneven surfaces and afford a better adhesion to the cement coating. The specimen is then immersed in water and soaked for 24 hours. When taken from the water, it is placed in a wooden mould constructed so that the specimen can be placed in an upright position with a space on each of the four sides. A mound of sand is formed on the top and held in place by a strip of wood first thoroughly soaked with water. The surface of the sand is covered with a piece of tissue paper to prevent the cement from flowing into it. The requisite quantity of sand is 1 000 g., and it must be fine, passing a No. 30 and being caught on a No. 40 sieve. On top of the sand is placed a 4-in. iron flange in which a ½-in. nipple, 4½ in. long, is screwed so as to make a tight joint in which there can be no leakage along the thread.

Neat cement, mixed to a paste about as stiff as can be handled and compacted conveniently, is placed in the mould, around the specimen and over the sand, thus forming a dome above it. About 83 lb. of cement and 21 lb. of water are used for the casing. When pouring the cement, four pieces of ½-in. galvanized ribbon wire, or some other form of reinforcing metal, are placed in pairs, two at right angles to the other two, over the top of the specimen and down along the side, as shown in Fig. 13, to reinforce the cement and prevent the dome from being blown off, which occurred in one of the experimental tests. As the cement is compacted, water rises to the surface, and bubbles of air come up through the cement and water. The casing is not considered complete until this bubbling ceases. The surface of the specimen, after it is complete, is covered with moist sand, and this is kept wet by a very slow flow from a water cooler. At the end of 24 hours, the neat cement has thoroughly set, and the specimen can be removed from the mould and buried in damp sand until ready to test.

Apparatus for Testing Permeability.—The apparatus for testing, together with the specimen itself, is shown in Fig. 2, Plate XI. The specimen is placed upon a tin funnel set in a wooden frame which rests upon any suitable foundation. The pipe projecting through the top of the specimen is connected by a union coupling with the bottom of the air-pressure tank, which is nearly filled with water before beginning the test. The pressure is raised by the hand-pressure air pump shown in the photograph, and main-

tained at any desired pressure for any required period. The time is read by a stop-watch reading for convenience to hundredths of minutes, instead of to seconds, which is started at the beginning of the tests and allowed to run as long as the experiment is continued. The water is caught in a bottle placed below the funnel under the specimen, and weighed at intervals by substituting a second bottle.

Recording the Data.—The method of recording the data of the tests is shown in a typical form in Table 16. Convenient periods of flow for most of the specimens were found to be 5-min. intervals, this giving sufficient time to weigh the bottle, and being frequent enough to obtain the average rates of flow. It was found that about 5 min. were required for the flow at any given pressure to become constant, and therefore the flow during the first 5 min. at any given pressure is not included in computing the average flow.

TABLE 16.—FORM FOR PERMEABILITY TESTS.

Specimen No. 165.—Maximum Size of Stone, $\frac{1}{2}$ in.—Analysis No. *378.*—Normal Mix: *Ellipse, 37% ordinate at diam. .105, Tangent to 100%.*—Per Cent. of Cement, *10%*.—Kind of Stone, *J. P.*—Kind of Sand, *J. P.*—Date Mixed, *Feb. 20.*—Date Beam Broken, *May 22.*—Age at Break, *91 d.*—Date of Coating, *May 23.*—Age at Perm. Test, *112 d.*—Dimensions before Coating, *5.91* × *5.95* × *18.0.**—Hours Soaked before Coating, *24.*—Time Valve Opened, *10.45.*

Times.	Pressure	Water passing, in grammes.	Rate of flow per minute, in grammes.	Remarks.
0	20 lb.	0		
10	" "	0	0	Pressure increased to 40 lb.
25	40 "	0		" " 60 "
29	60 "			Water appeared.
35	60 "	61		
40	" "	98		
45	" "	103	20	
50	" "	99		
55	80 "	150		Pressure increased to 80 lb.
60	" "	177		
65	" "	189	36	
		168		

GENERAL REMARKS.

Water appeared after a pressure of 20 lb. had been applied for 10 min., of 40 lb. 15 min., and 60 lb. 4 min., making specimen under increasing pressures 29 min. before water appeared.

*This measurement is assumed to be 18 in. Other dimensions taken before surface was roughened.

Adhesion of Cement Casting to Concrete Block.—The only possibility of a leakage which is not immediately apparent is between

the specimen and the neat-cement casing. In order to prove that the adhesion was perfect, several specimens were broken after the completion of the tests. One of the first specimens showed signs of slight percolation between the specimen and the coating, but by more carefully roughening the surface before coating, this was entirely prevented in subsequent tests, and the bond of union was found to be absolutely perfect, the adhesion of the cement casing to the concrete being stronger than the adhesion of the ingredients of the concrete.

Results of Permeability Tests.—An examination of Table 18 reveals a number of points, which stand out very clearly in the results, and form the basis for the conclusions on the following pages. To establish definite laws, and compare materials not included in this series of tests, further experiments are essential. The permeability of the different specimens is indicated in Columns 18 to 21 by the rate of flow, in grammes per minute, at different pressures. The specimens are all of approximately the same length, namely, 17 in., and the flow is through this total length.

TABLE 17.—EFFECT OF PERCENTAGE OF CEMENT ON PERMEABILITY OF CONCRETE BEAMS OF VARIOUS AGGREGATES HAVING A MAXIMUM DIAMETER OF 2¼ INCHES.

No. of Beam.	Proportion, by weight.	Percentage of cement to total dry material.	Kind of Material.		Time in which water appears, in minutes.	Rate of Flow of Water, in Grammes per Minute, at the Following Pressures, per Square Inch.			
			Stone.	Sand.		20 lb.	40 lb.	60 lb.	80 lb.
193	1:11.5	8.0	J. P.*	J. P.	7	25	161	237	273
155	1:9	10.0	"	"	3	11	24	369	49
179	1:7	12.5	"	"	3	15	22	30	38
183	1:5.75	15.0	"	"	5.5	5	8	10	12
233	1:10.8	8.5	C. B.	C. B.	3	15	25	38	43
217	1:8.4	10.6	"	"	17	1	3	5	6
224	1:6.5	13.0	"	"	100	0	0	0	0.5
225	1:5.3	15.9	"	"	98	0	0	0	1.4
213	1:8.8	10.2	J. P.	C. B.	9	4	11	17	22
218	1:6.88	12.7	"	"	10	2	2	3	3
216	1:5.54	15.6	"	"	.....	0	0	0.7	1.4

*J. P. = Jerome Park; C. B. = Cowe Bay.

Effect of Percentage of Cement Upon Permeability.

Conclusion 17.

The data in Table 18 which bear on this subject have been rearranged in Table 17. It is seen that the rate of flow is less as the percentage of cement increases, and in a much larger inverse ratio. The most extreme differences are shown in the straight Jerome Park materials, with which the rate of flow at 80 lb. pressure decreases from 273 g. per min. in the specimens having 8% of cement, to 12 g. per min. in the specimen having 15% of cement.

Effect of Size of Stone Upon Permeability.

Conclusion 18.

The data relating to this subject in Table 18 have been rearranged for more convenient reference in Table 19. It is seen that the evidence is conflicting, the mixtures with Cowe Bay material indicating that the rate of flow is less as the maximum size of the aggregate is greater, while the mixtures of Jerome Park material are erratic, the 1-in. and ½-in. stone giving about the same rate of flow, and the 2½-in. stone giving greater than either of the smaller sizes, but the results are much less definite than those of the other tests mentioned. All the straight Jerome Park specimens, in fact, are more erratic, as regards permeability, than the specimens containing more rounded grains which can be more uniformly mixed.

Decrease of Permeability with Age.—The rate of flow decreases materially with age. The difference in the age of the specimens in Table 18 is not sufficient to draw conclusions, but a comparison of these tests with those in Table 21, which were made under similar conditions, indicate a very great reduction in the flow from that at 1 month, to that at 3½ months.

Increase of Permeability with Pressure.—By the comparison in Table 18, of the values in Columns 19, 20 and 21, it is seen that the rate of flow increases nearly uniformly with the increase of pressure.

Effect of Thickness of Concrete Upon Permeability.—The data in Table 20 indicate that the rate of flow increases as the thickness of the concrete decreases, but in a much larger inverse ratio. All the tests in this table were made upon specimens from the same beam. Column 4 gives the results from a regular specimen

TABLE 18.—PERMEABILITY OF 6 × 6 × 17-IN. CONCRETE PORTLAND CEMENT.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
Reference numbers.	Proportion, by weight.	Percentage of cement to total dry material.	Kind of stone.	Kind of sand.	Maximum size of stone, in inches.	Uniformly graded above given diameter, in inches.	Percentage finer than diameter in Column 7.*	Curve below diameter in Column 7.
155	1:9	10	J. P.	J. P.	2½	0.20	37.80	Ellipse.
151	1:9	10	"	"	1	0.10	35.90	"
165	1:9	10	"	"	¼	0.075	36.30	"
168	1:2½:6½	10	"	"	2½	.....	.....	.....
167	1:3:6	10	"	"	2½	.....	.....	.....
172 B	1:2½:6½	10	"	"	1	.....	.....	.....
170	1:3:6	10	"	"	1	.....	.....	.....
192 B	1:2½:6½	10	"	"	¼	.....	.....	.....
176 B	1:3:6	10	"	"	¼	.....	.....	.....
187 A	1:9 uniform stone.	10	"	"	2½	0.20	37.8	Ellipse.
193 C	1:11.5	8	"	"	2½	0.20	37.8	"
179 B	1:7	12.5	"	"	2½	0.20	37.8	"
183 B	1:5¼	15	"	"	2½	0.20	37.8	"
217	1:8.4	10.6	C. B.	C. B.	2½	0.20	32.1	"
221	1:2.91:5.62	10.6	"	"	2½	.....	.....	.....
234	1:8.4	10.6	"	"	2½	0.20	37.8	Ellipse.
227	1:2.91:5.62	10.6	"	"	1	.....	.....	.....
233	1:10.8	8.5	"	"	2½	0.20	32.1	Ellipse.
224	1:6.5	13.75	"	"	2½	0.20	32.1	"
225	1:5.3	15.9	"	"	2½	0.20	32.1	"
213 C	1:8.8	10.2	J. P.	"	2½	0.20	33.8	"
219 B	1:8.8	10.2	"	"	2½	0.20	37.8	"
205	1:2.92:5.88	10.2	"	"	2½	.....	.....	.....
203 C	1:2.92:5.88	10.2	"	"	1	.....	.....	.....
207 C	1:2.92:5.88	10.2	"	"	½	.....	.....	.....
218	1:6.88	12.7	"	"	2½	0.20	35.8	Ellipse.
216	1:5.54	15.3	"	"	2½	0.20	33.8	"

* Including cement.

† In Cowe Bay mixtures the percentage of cement by weight is increased in Park material.

J. P. = Jerome Park; C. B. = Cowe Bay.

PRISMS, MADE OF VARIOUS AGGREGATES AND GIANT AGE, 3½ MONTHS.

(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)
CALCULATED VOLUME, IN CUBIC FEET OF MATERIAL IN 1 CU. FT. OF BEAM, AS MADE.								RATE OF FLOW OF WATER, IN GRAMMES PER MINUTE.			
Cement.	Aggregate.	Total dry.	Water.	Total voids.	Total calculated volume.	Age, in days.	Time in which water appears, in minutes.	At 20 lb. per sq. in.	At 40 lb. per sq. in.	At 60 lb. per sq. in.	At 80 lb. per sq. in.
0.078	0.777	0.855	0.173	0.145	1.028	110	3	11.2	24.4	36.9	48.7
0.075	0.745	0.820	0.191	0.180	1.011	112	21	0	11.6	21.6	35.1
0.070	0.700	0.770	0.208	0.230	0.978	112	29	0	.....	20	36
0.076	0.758	0.824	0.154	0.166	0.988	119	20	0	3	8	12
0.074	0.738	0.812	0.144	0.188	0.956	113	3	15	27	46	.....
0.074	0.736	0.810	0.232	0.190	1.042	105	3.5	25	87	153	....
0.070	0.730	0.802	0.226	0.198	1.028	119	7	8	18	31	49
0.070	0.703	0.773	0.224	0.227	0.997	137	25	0	13	35	63
0.070	0.702	0.772	0.199	0.228	0.971	101	24	0	19	34	50
0.077	0.775	0.852	0.154	0.148	1.006	119	3.75	14	21	27	33
0.060	0.771	0.831	0.170	0.169	1.001	140	7	25	161	237	273
0.037	0.755	0.852	0.189	0.148	1.041	107	3	15	22	30	38
0.130	0.715	0.845	0.186	0.155	1.031	105	5.5	5	8	10	12
0.079	0.776	0.855	0.133	0.145	0.988	146	17	1	3	5	6
0.076	0.750	0.826	0.157	0.174	0.983	133	134	0	0	0	0.7
0.079	0.781	0.860	0.147	0.140	1.007	147	72	0	0	1.9	3.9
0.077	0.755	0.832	0.173	0.168	1.005	150	125	0	0	0	1.8
0.064	0.807	0.871	0.154	0.129	1.025	147	3	14.7	25	37.6	42.7
0.100	0.765	0.865	0.149	0.135	1.014	151	100	0	0	0	0.5
0.129	0.738	0.867	0.147	0.133	1.014	151	98	0	0	0	1.4
0.080	0.793	0.873	0.152	0.127	1.025	137	8.75	4	11	17	22
0.078	0.772	0.850	0.160	0.150	1.010	116	45.5	0	1	2	3
0.075	0.740	0.815	0.160	0.185	0.975	132	7	1	4	8	12
0.075	0.745	0.820	0.156	0.180	0.976	132	26	0	5	10	15
0.070	0.703	0.773	0.195	0.227	0.968	132	29	0	10	17	25
0.099	0.767	0.867	0.151	0.133	1.017	148	10.5	2	2	3	3
0.117	0.735	0.852	0.154	0.148	1.006	147	.....	0	0	0.7	1.4

each case to balance the difference in specific gravity between this and the Jerome

TABLE 19.—EFFECT OF SIZE OF STONE ON PERMEABILITY OF CONCRETE BEAMS.

No. of Beam.	Proportion, by weight.	Percentage of cement to total dry material.	Kind of Aggregate.		Maximum size of stone, in inches.	Time in which water appears, in minutes.	Rate of Flow of Water, in Grammes per Minute, at the Following Pressures per Square Inch.			
			Stone.	Sand.			20 lb.	40 lb.	60 lb.	80 lb.
155	1:9	10	J. P.	J. P.	2¼	3	11	24	369	49
151	:9	10	"	"	1	21	0	12	22	35
165	1:9	10	"	"	½	29	0	..	20	36
168	$1:2^5:6^5$	10	J. P.	J. P.	2¼	20	0	3	8	12
172	$1:2^5:6^5$	10	"	"	1	3.5	25	87	153	..
192	$1:2^5:6^5$	10	"	"	½	2.5	0	13	35	63
167	1:3:6	10	J. P.	J. P.	2¼	3	15	27	46	..
170	1:3:6	10	"	"	1	7	8	18	31	49
176	1:3:6	10	"	"	½	24	0	19	34	50
221	$1:2^{91}:5^{82}$	10.6	C. B.	C. B.	2¼	134	0	0	0	0.7
227	$1:2^{91}:5^{82}$	10.6	"	"	1	125	0	0	0	1.8
205	$1:2^{92}:5^{08}$	10.2	J. P.	C. B.	2¼	7	1	4	8	12
203	$1:2^{92}:5^{08}$	10.2	"	"	1	26	0	5	10	15
207	$1:2^{92}:5^{08}$	10.2	"	"	½	29	0	10	17	20

J. P. = Jerome Park; C. B. = Cowe Bay.

TABLE 20.—COMPARATIVE PERMEABILITY OF CONCRETE THROUGH A LENGTH OF 8.5 INCHES *vs.* A LENGTH OF 17 INCHES. SPECIMEN NO. 224. COWE BAY AGGREGATES. PROPORTIONS, 1 : 6.5 (BY WEIGHT). AVERAGE AGE, 153 DAYS.

(1)	(2)	(3)	(4)	(5)
Pressure, in pounds per square inch.	Rate of Flow, in Grammes per Minute.			
	8.5 × 5.95 × 6.05-in. specimens.			17 × 5.95 × 6.15-in. specimen.
	(1)	(2)	Average.	
20	0.16	0.47	0.32	0
40	0.87	1.49	1.18	0
60	1.40	2.58	1.99	0
80	2.17	3.93	3.05	0.47

17 in. in length and Columns 2 and 3 the results from two specimens, each 8½ in. long, from the same beam. The difference in the permeability of 1 and 2 is greater than would be expected from two similar specimens, but this is probably explained by differences

in homogeneity due to the large size of stone used for such a short specimen, and to the fact that the length of the two pieces which were broken from larger ones may not have been absolutely the same. Evidently, further comparative tests of similar specimens are required.

Rate of Flow During a Four-Hour Period.—A specimen subjected to continuous uniform pressure for a period of 4 hours shows a practically uniform rate of flow during this time. The results are given in Table 21.

TABLE 21.—Uniformity of Rate of Flow through a Specimen of 1 : 2.5 : 6.5 (by Weight). Concrete Specimen No. 168. 5.8 × 5.9 × 17.0 Inches Long. Jerome Park Screenings and Stone. Age, 119 Days.

(1) Time, in hours and minutes.	(2) Pressure, in pounds per square inch.	(3) Water passing, in grammes.	(4) Rate of flow per minute, in grammes.	(5) Rate of flow per hour, in grammes.
0	60	0	.....	.....
1:00	"	446	.....	446
1:05	"	42	8.4	.....
1:10	"	40	8.0	.....
1:15	"	41	8.2	.....
2:00	"	344	7.6	467
2:05	"	40	8.0	.....
2:10	"	40	8.0	.....
2:15	"	40	8.0	.....
3:00	"	338	7.5	458
3:05	"	40	8.0	.....
3:10	"	39	7.8	.....
3:15	"	40	8.0	.....
4:00	"	347	7.7	466

Volumetric Tests of Density of Neat Cement and Mortar.

During the progress of the experiments at Jerome Park, frequent tests were made of the density of the cement which was being used, and also of the densities of various mortars. For this purpose, the cylinder apparatus described for the density experiment upon concrete was used; tests were also made upon a smaller scale, with a 200-c. c. graduate for the mould. The apparatus used for these tests on the small scale, and the Jackson specific gravity flask, also constantly used, are shown in Fig. 1, Plate XIV.

Effect of Different Percentages of Water Upon Volume and Density of Portland Cement Mortars and Concretes.

Table 22 and Fig. 2, Plate XIV, illustrate the variation in volume of paste produced from the same weight of the same cement with different percentages of water. They also show the difference in volume between the fresh paste as mixed and the same paste after standing for a period of about 3 hours and being compacted by occasionally shaking and tapping the mould. The tests were made in the graduates, as previously described. By adopting a standard method in all the tests, and maintaining a uniform consistency, good results were obtained. When sand is mixed with the cement, there is much less variation in the volumes of the mortars due to different percentages of water than in neat pastes, but the variation is generally very appreciable, although less with coarse than with fine sands.

TABLE 22.—Effect of Different Percentages of Cement Upon the Volume of Neat Portland Cement Paste.

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
								Absolute Volume.			
						Volume of Paste.		Fresh.		Finally compacted.	
Experiment number.	Nominal mix.	Kind of cement.	Weight of cement, in grammes.	Water used in mixing, in grammes.	Percentage of water used in mixing.	Fresh, in cubic centimeters.	Finally compacted, in cubic centimeters	Water.	Cement (density).	Water.	Cement (density).
								w.	*c.*	*w.*	*c.*
463	Neat.	Giant.	300	60	20	165.8	163.8	0.362	0.584	0.354	0.590
464	"	"	300	69	23	168.7	167.7	0.391	0.546	0.404	0.574
465	"	"	300	78	26	174.0	172.3	0.438	0.543	0.441	0.560
467	"	"	300	96	32	183.2	179.3	0.504	0.568	0.508	0.534
447	"	"	300	150	50	244.0	221.0	0.613	0 395	0.571	0.435
449	"	"	300	300	100	390.6	214.3	0.772	0.248	0.569	0.436

The resulting volumes of different concretes, of the same materials and proportions, but with varying percentages of water are nearly constant, provided sufficient water is added to permit thorough compacting. When an excess of water is used, it is nearly all expelled as the solid materials settle by gravity. There is, however, a slight variation even in concrete. A dry mixture, pro-

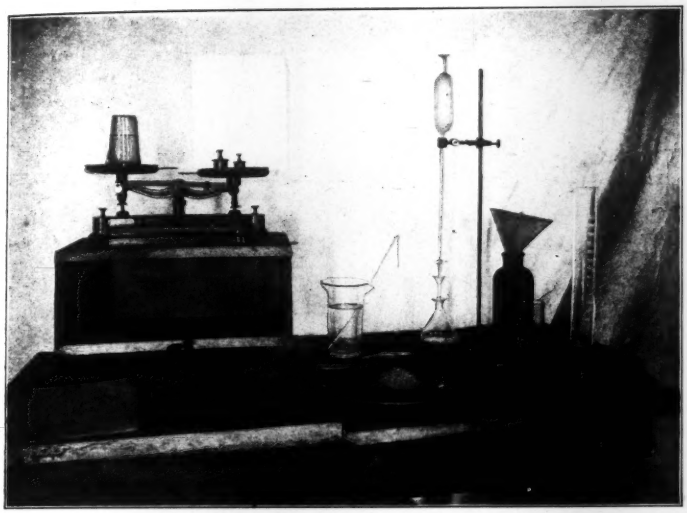


FIG. 1.—APPARATUS FOR VOLUMETRIC TESTS OF DENSITY OF CONCRETE.

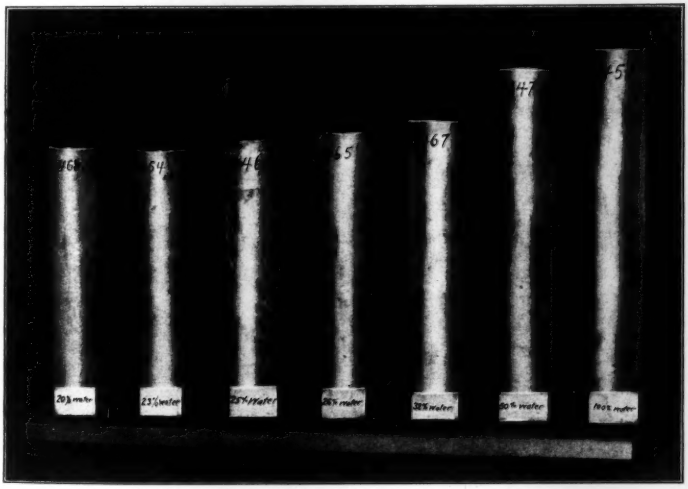


FIG. 2.—VARIATION IN VOLUME OF PASTE PRODUCED BY MIXING THE SAME WEIGHT OF CEMENT WITH DIFFERENT PERCENTAGES OF WATER.

vided sufficient water is added to rise to the top on hard ramming, can be compacted somewhat more than a concrete of medium consistency, while a very wet concrete is apt to occupy a very slightly larger volume, and hence be less compact than a medium concrete. In most cases, the difference between the medium and very wet mixtures is only noticeable by extremely accurate measurements.

Elasticity Tests.

Figs. 14, 15, and 16 show a number of typical results of tests of elasticity of several of the capped prisms. These tests were made at the Stevens Institute. Readings of compressive deformations were taken at increments corresponding to unit increments of 100 lb., and in most of the tests the load was released at 500 and 1 000 lb. and the permanent set recorded. In a few cases, shown by zigzag lines, the effect of repeated stress was also measured. The results are given in Table 23.

It should be noticed in the diagrams that nearly all the curves run up sharply from 100 lb., the first load applied, indicating that the modulus between 100 and 200 lb. is not greater than at the next higher loadings. After deducting the set, the deformation (shown by the curve labeled "Elastic"), in the majority of the specimens, at first increases uniformly, the curve being in these cases a straight line in its lower portion. The point of tangency with the decided curve is sometimes considered the elastic limit of the concrete.

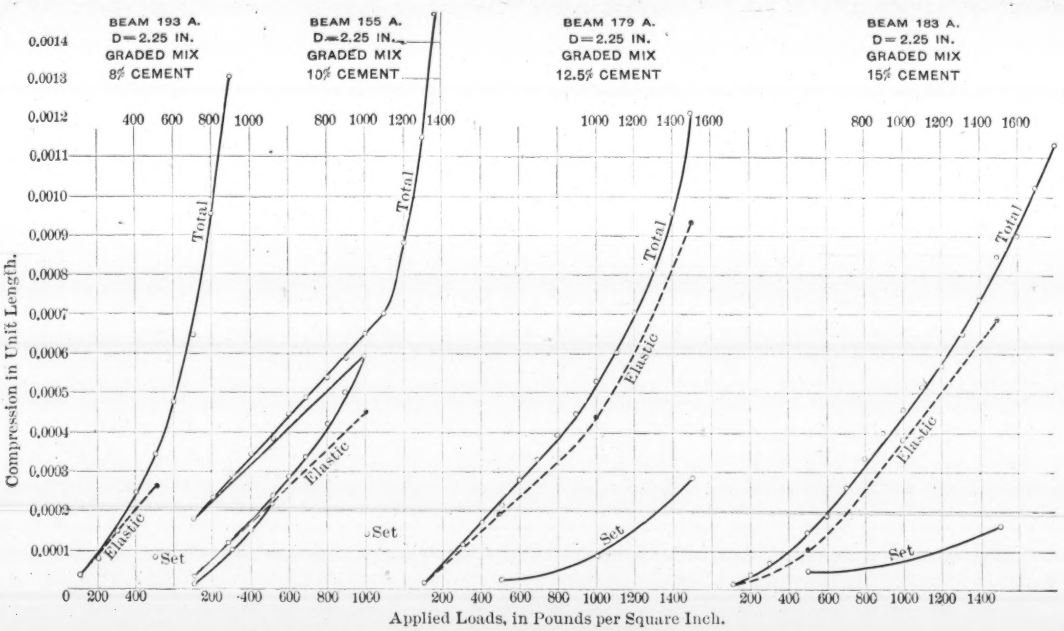
The table shows several interesting comparisons:

1.—The modulus increases with the maximum size of stone. Compare, respectively, specimens 1, 2, 3; 5, 6, 7; 17, 18; 22, 24.

2.—The modulus increases with the percentage of cement. Compare, respectively, 8, 9, 10, 11; 13, 14, 15, 16; 19, 20, 21. There are one or two exceptions to this rule, but the general trend is unquestionable.

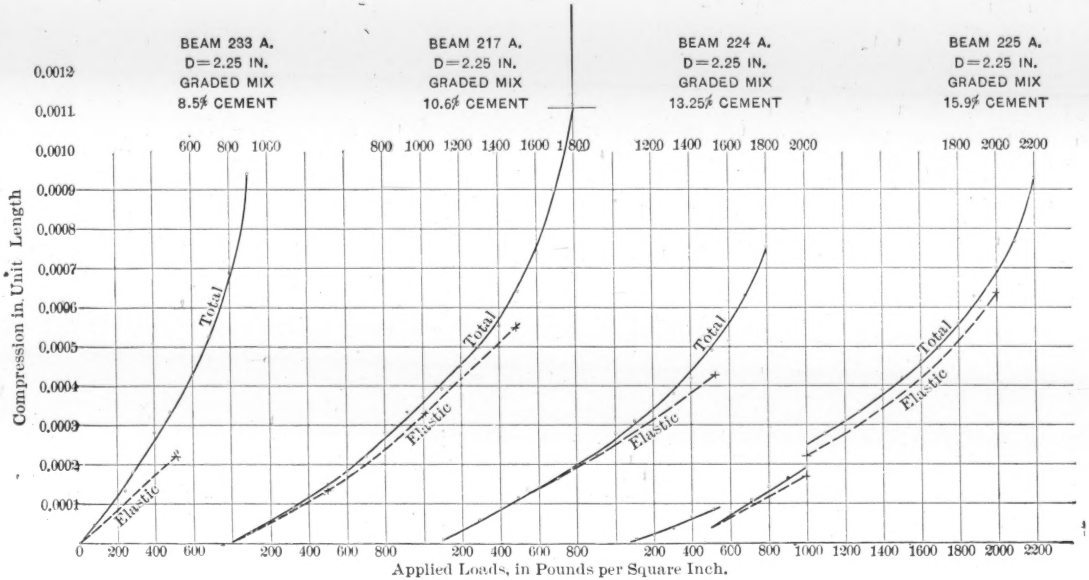
3.—In general the modulus of the Cowe Bay gravel and sand is higher than the modulus of similar specimens mixed with Jerome Park sand and screenings.

4.—The modulus of specimens with a mixture of Jerome Park stone and Cowe Bay sand is in general slightly lower than similar specimens of straight Cowe Bay material.



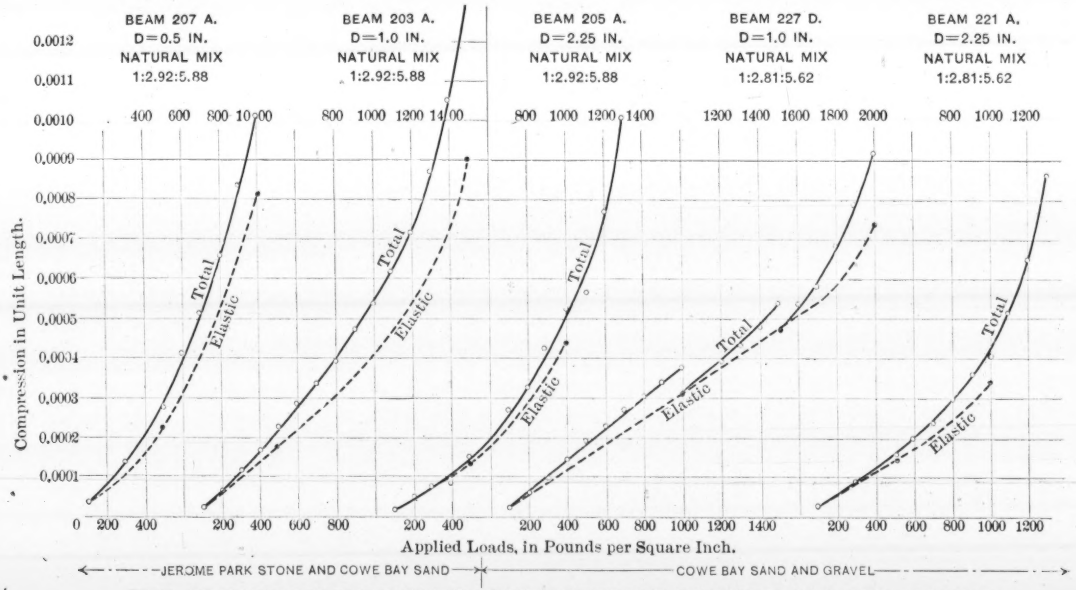
TOTAL AND ELASTIC COMPRESSION OF 6 BY 6 BY 18 IN. CONCRETE PRISMS MADE OF GIANT PORTLAND CEMENT AND JEROME PARK SCREENINGS AND BROKEN STONE. AGE, ABOUT 140 DAYS.

FIG. 14.



TOTAL AND ELASTIC COMPRESSION OF 6 BY 6 BY 18-IN. CONCRETE PRISMS MADE OF GIANT PORTLAND CEMENT AND COWE BAY SAND AND GRAVEL. AVERAGE AGE, ABOUT 140 DAYS.

FIG. 15.



TOTAL AND ELASTIC COMPRESSION OF 6 BY 6 BY 18-IN. CONCRETE PRISMS MADE OF GIANT PORTLAND CEMENT AND VARIOUS AGGREGATES. AGE, ABOUT 140 DAYS.

FIG. 16.

TABLE 23.—MODULUS OF ELASTICITY OF CONCRETE PRISMS.

Tests for deformation. Jerome Park Reservoir. 1905.

(1) Reference number.	(2) Beam number.	(3) Proportion, by weight.	(4) Percentage of cement to total dry material.	(5) Kind of stone.	(6) Kind of sand.	(7) Maximum size of stone, in inches.	(8) Uniformly graded above diameter, in inches.	(9) Percentage finer than diameter in Column 8.	(10) Density.	(11) Ultimate strength, in pounds per square inch.	(12) Modulus of elasticity, in pounds per square inch.	(13) Modulus taken between limits, in pounds per square inch.
1	155A	1:9	10	J. P.	J. P.	2.25	0.20	37.8	0.855	1 425	2 143 000	100—1 000
2	151A	1:9	10	"	"	1.00	0.10	35.9	0.820	1 085	1 702 000	100— 500
3	165A	1:9	10	"	"	0.50	0.075	36.3	0.770	960	1 436 000	100— 500
4	192A	$1:2^{3}:6^{5}$	10	"	"	0.50	Natural Mix		0.773	875	1 274 000	300— 500
5	167	1:3:6	10	"	"	2.25	"	"	0.812	965	2 425 000	100— 500
6	170A	1:3:6	10	"	"	1.00	"	"	0.802	860	1 798 000	100— 500
7	176A	1:3:6	10	"	"	0.50	"	"	0.772	670	878 000	200— 500
8	198A	$1:1^{5}$	8	"	"	2.25	0.20	37.8	0.891	940	1 750 000	100— 500
9	155A*	1:9	10	"	"	2.25	0.20	37.8	0.855	1 425	2 143 000	100—1 000
10	179A	1:7	12.5	"	"	2.25	0.20	37.8	0.852	1 710	2 250 000	200— 800
11	183A	$1:5^{67}$	15	"	"	2.25	0.20	37.8	0.845	1 920	4 650 000	100— 500
12	234D	$1:8^{13}$	10.6	C. B.	C. B.	2.25	0.20	37.8	0.860	1 800	2 867 000	100—1 500
13	233A	$1:10^{76}$	8.5	"	"	2.25	0.20	32.1	0.871	940	2 252 000	100— 500
14	217A	$1:8^{13}$	10.6	"	"	2.25	0.20	32.1	0.855	1 880	3 920 000	100— 500
15	224A	$1:6^{51}$	13.25	"	"	2.25	0.20	32.1	0.865	2 110	3 675 000	100—1 000
16	225A	$1:5^{30}$	15.9	"	"	2.25	0.20	32.1	0.867	2 315	4 273 000	500—1 000
17	221A	$1:2^{81}:5^{62}$	10.6	"	"	2.25	Natural Mix		0.826	1 455	3 260 000	100— 600
18	227D	$1:2^{81}:5^{62}$	10.6	"	"	1.00	"	"	0.832	2 065	3 100 000	100—1 500
19	213A	$1:8^{80}$	10.2	J. P.	"	2.25	0.20	33.8	0.873	2 000	3 480 000	100— 500
20	218A	$1:6^{87}$	12.75	"	"	2.25	0.20	33.8	0.866	2 150	3 830 000	100— 800
21	216A	$1:5^{54}$	15.3	"	"	2.25	0.20	33.8	0.852	2 440	3 508 000	200—1 000
22	205A	$1:2^{92}:5^{88}$	10.2	"	"	2.25	Natural Mix		0.831	1 480	3 037 000	300— 500
23	203A	$1:2^{92}:5^{88}$	10.2	"	"	1.00	"	"	0.820	1 555	2 550 000	100— 700
24	207A	$1:2^{92}:5^{88}$	10.2	"	"	0.50	"	"	0.783	1 145	2 206 000	100— 400

* Beam 155 repeated here to facilitate comparison.
J. P. = Jerome Park; C. B. = Cowe Bay.

In conclusion the writers desire to express to their assistants in this work their hearty appreciation of their interest and zeal, without which co-operation the results accomplished with the time and funds available would have been impossible. Especial thanks are due to Thomas H. Wiggin, Assoc. M. Am. Soc. C. E., for his valued advice and assistance in starting the experiments; to William Hauck, Assoc. M. Am. Soc. C. E., for his careful attention to the large number of calculations involved in the tables of this paper, and to James L. Davis, Assoc. M. Am. Soc. C. E., and Messrs. Charles M. Montgomery, and William E. King, for their faithful labors in connection with the tests.

DISCUSSION.

Mr. Davis. JAMES L. DAVIS, ASSOC. M. AM. SOC. C. E.—As the methods given in this paper are presumably new to many, it may be of some interest to show their application to aggregates other than the two used in the experiments on which the authors' conclusions were based.

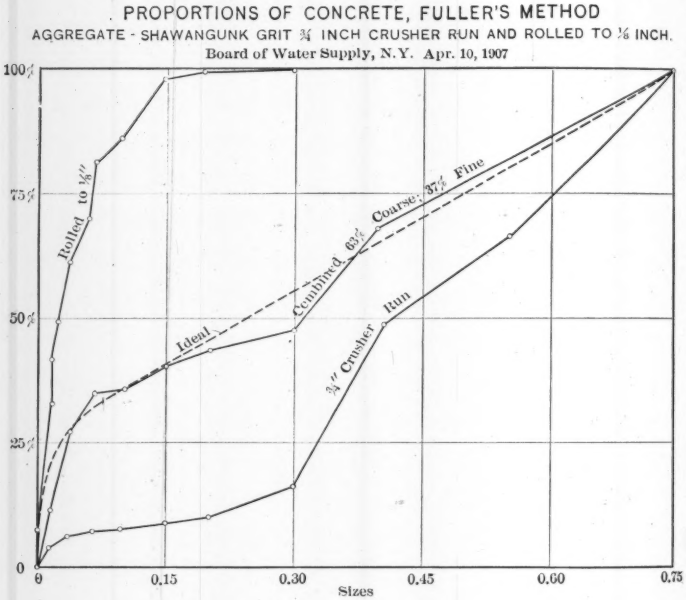


FIG. 17.

The Board of Water Supply of the City of New York, in its work of bringing an additional supply of water from the Catskills, has established a laboratory for the investigation of the chemical and physical problems which may come up in connection with the works, and it has been the speaker's duty to collect and examine a large number of aggregates which may be available for construction purposes at various points along the route of the proposed aqueduct.

Mr. Davis.

What is here presented contains nothing original as to method, but is simply the application of the methods proposed in this paper to a few aggregates as they have been found. These methods are shown by the diagrams, Figs. 17 to 21, inclusive. The calculations used in combining the curves have been omitted. The methods by which this is done are not difficult, and are very fully explained in "Concrete, Plain and Reinforced," by Messrs. Taylor and Thompson, in the chapter on proportioning concrete written by Mr. Fuller.

The diagram, Fig. 17, is a study of crushed rock, the mechanical analysis having been made and plotted in the manner previously described. This rock is the Shawangunk grit, which is a quartzite.

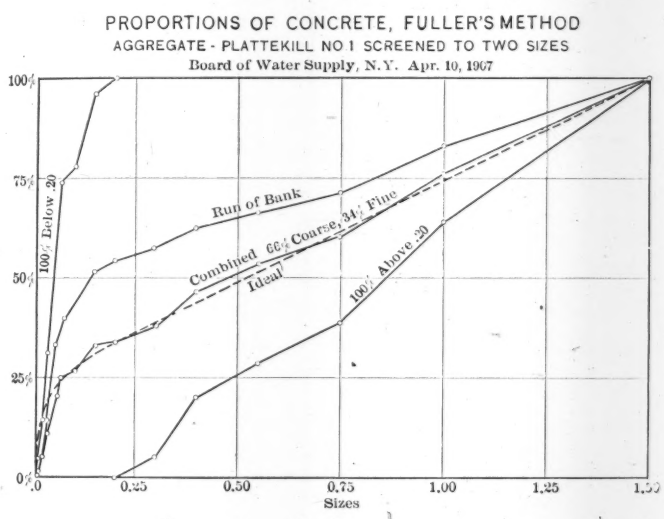


FIG. 18.

The lower curve shows the analysis of the material as it came from the crusher, ranging in size from $\frac{3}{4}$ in. down. The upper curve shows the same material run through rolls set $\frac{1}{8}$ in. apart.

The ideal curve is shown as a dotted line. This curve was plotted by using the constants given for Jerome Park crushed stone, that stone being taken as representative of crushed rock in general, as far as the constants are concerned. The combined curve shows the result of combining the two sizes of materials in the proportions of 63% coarse and 37% fine. The resulting curve conforms closely to the ideal down to stone of 0.4 in. diameter. From this size down to that of 0.15 in. diameter, there is a deficiency. Of

Mr. Davis.

course, this could be remedied by combining the different proportions, which would raise the combined curve.

One of the conclusions of the paper, however, is that a deficiency in the medium-sized stone will not result in a loss of strength in the concrete, while an excess will cause such a loss; therefore, it is better not to change the proportions. The combined curve also shows a deficiency in the quantity of fine sand, but this will be made up by the addition of the cement, as the ideal curve includes the cement; and, with the cement added, there will be a new or corrected combined curve, for the sand portion, which will approach the ideal. This case is a very ordinary one.

Fig. 18 shows a gravel. The run of the bank is plotted, the lower and upper curves showing the result of screening this material to two sizes—that which is retained on a sieve of 0.2-in. mesh. or clear opening, and that which passes such a sieve.

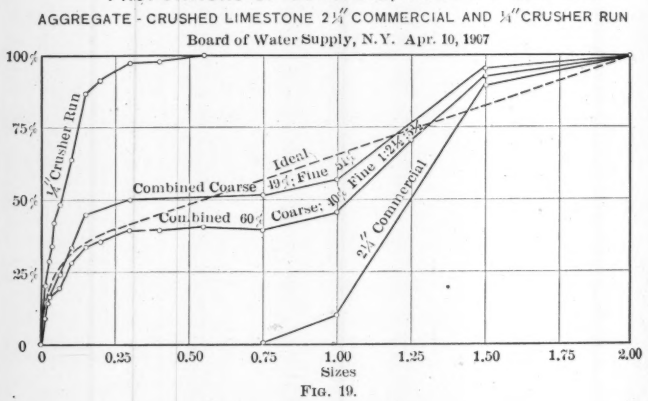


FIG. 19.

The ideal curve was plotted with the constants given for Cowe Bay gravel, and shows, by the length of the ordinate at 0.2-in. diameter, that 66% of the ideal mix should be coarser than this. The combined curve shows the result of taking this percentage of coarse material and 34% of fine material. It approaches very closely the ideal mix. There is a deficiency of fine sand, but, as in the case of the Shawangunk grit, this would be made up by the cement.

Fig. 19 shows a crushed limestone, available in the New York market. The lower curve shows the product sold as 2¼-in. stone. It ranges, actually, from 2 in. to ¾ in. Such stone is often used,

with screenings, as a substitute for sand. The upper curve shows such a material produced by a crusher with jaws set to the maximum opening of ¼ in. Mr. Davis.

In these experiments, the two combined curves show how, by scientific study, improvement may be made over the usual rule-of-thumb methods of mixing. Suppose a concrete having the proportions, 1 cement to 8 aggregate, is wanted. In ordinary work a common proportion would be 1 : 2½ : 5½. This is equivalent, in this case, to 60% of coarse and 40% of fine aggregate. The plotting shows a deficiency in all sizes below those of about 1¼ in. diameter. An improvement is shown by the upper combined curve in proportions of 49% coarse and 51% fine aggregate, and the case involves

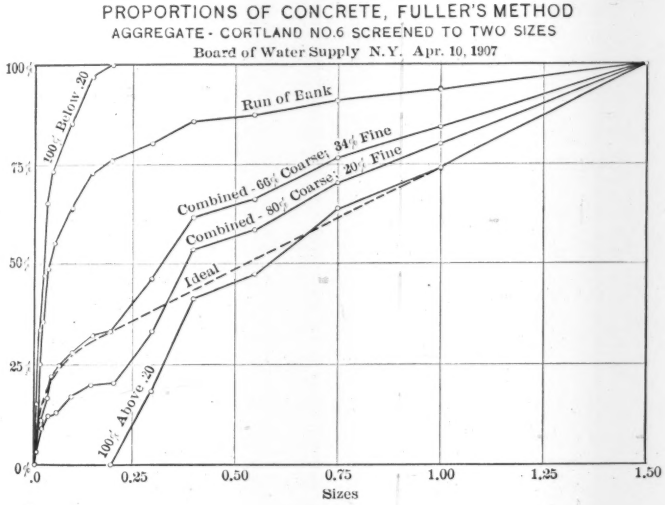


FIG. 20.

no extra expense for labor, but the cost might be increased somewhat by the use of the increased proportion of fine material, as it is more expensive to crush it to this fine size.

Fig. 20 shows a gravel from Cortland, N. Y. It was not selected because it illustrated particularly the application of this method of proportioning, but because it is one of the best gravels to be found in the vicinity where it occurs.

Concrete is often made by mixing a certain proportion of cement with gravel, as it comes from the bank. A comparison of the curve of the run of the bank with the ideal curve shows how far from good the results would be in such a case.

Mr. Davis.

The upper and lower curves show the analyses of this material screened to two sizes, as shown in a preceding case. By the length of the ordinate, at 0.2 in. diameter, the ideal curve shows that 66% of the mix should be coarser than this size.

The upper combined curve shows the result. This gives almost ideal results for the sand portion, but the proportions passing various medium-sized screens are altogether too large. The lower combined curve shows the result of attempting to remedy this by using a larger proportion of the coarse material. The stone portion of the mix is improved, but the sand portion becomes totally unfit. This demonstrates that this material screened to these two particular sizes cannot be combined in ideal proportions.

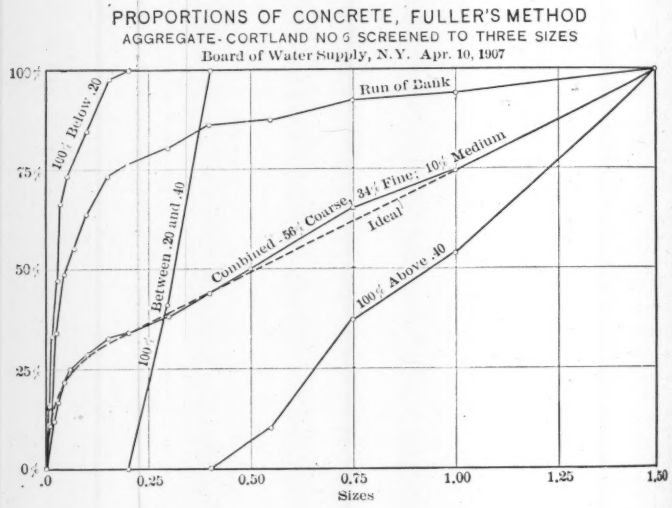


FIG. 21.

Inspection of the curve shows that the material retained above the 0.2-in. diameter conforms closely to the ideal down to 0.4-in. diameter, and that the trouble with the combined curves occurs between 0.2 and 0.4 in. diameter. This suggests screening the material into three instead of two sizes. The result is shown by Fig. 21, on which the run of the bank, the ideal curve, and the material below the 0.2-in. size are plotted, as in Fig. 20. The coarse material has been separated into two sizes, ranging from 0.4 in. up, and from 0.2 to 0.4 in.

The combined curve shows the result of combining the three sizes in proportions of 56% coarse, 34% fine, and 10% medium. The result is a close conformation to the ideal curve, and it might be safely stated that the resulting concrete would be strong and economical. Mr. Davis.

It would not be at all impracticable to use these proportions, as it would only be necessary to interpose another screen of about ⅜-in. mesh, when screening out the sand from the run of the bank.

The authors explain the methods of making laboratory comparisons of the different aggregates and studying the various mixtures, but these do not conflict with the more common ones used for determining the proportions of voids in the aggregate; they simply accomplish the same result in a more accurate and scientific way.

H. de B. Parsons, M. Am. Soc. C. E.—The speaker would like to ask the following questions: Mr. Parsons.

Did the experiments show any limit to the tensile stress to which concrete might be subjected with safety? There are masonry structures in which the masonry oftentimes is under tension, but do the experiments show any limit in that direction?

Did age affect the concrete in any way? Frequently, a cement which has a certain strength at the end of a given time, say 1 month, or 2 months, is not as strong at the end of an extended period as another cement which is weaker at the end of the same first period of 1 month, or 2 months. Has this difference in strength any marked effect when the aggregate is mixed as suggested in the paper? Were any experiments made along this line?

Was the effect of heat studied? In other words, would a concrete mixed according to the ideal curve resist heat any better than a concrete in which the same materials were used, but not mixed according to the ideal curve?

Allen Hazen, M. Am. Soc. C. E.—All who use concrete are interested in the art of selecting and mixing the ingredients in such a way as to get the best results; and a better understanding of the underlying principles is certain to be helpful in many cases in obtaining a better utilization of available materials. Mr. Hazen.

There is some comfort to be drawn from the paper, and especially from Mr. Davis' discussion of it, in regard to the methods which have been in use. Mr. Davis, in his diagrams, has designated the various materials as "coarse" and "fine," and it is noted that, as he computes, he gets the best results in nearly all cases when about one-third of the whole volume is made up of fine material. Now, one-third of sand is not very far from the proportion which is generally used in making concrete, and it appears that this analysis does not tend to change this proportion.

The authors contend that, by using a theoretically correct mix-

Mr. Hazen. ture—and by theoretically correct is meant the kind of a mixture which they describe—about 12% of the cement can be saved, as compared with a concrete mixed in the ordinary way from average materials. That is to say, with concrete mixed in the usual way, from good materials, but without any special precaution, it would take 12% more cement to produce a concrete of the same strength as could be obtained by the use of especially prepared and selected mixtures.

Now, in mixing ordinary concrete, at current prices for materials, and with an average mixture, 12% of the cement probably represents 25 cents per cu. yd.; and the question is raised as to whether the additional grading, mixing, and supervision required to get the ideal mixture can be carried out for as little as 25 cents per cu. yd. In other words, can the saving in cement be made to pay for the extra trouble in connection with the other materials? Obviously, this will depend upon conditions. There will certainly be many cases where an improvement in the mixture can be obtained at an expense so small that the saving in cement will more than justify it; but there will be other cases, and many of them, where it will be cheaper and better to use the additional cement than make the extra effort to secure what the authors consider an ideal arrangement of the other ingredients.

The paper certainly will help to an understanding of what is desirable; and "knowing what is wanted" is a long step toward securing it. The authors have certainly helped in that direction, and are to be congratulated.

Mr. Wagner. SAMUEL TOBIAS WAGNER, M. AM. SOC. C. E. (by letter).—This paper is of scientific interest, and is particularly valuable on account of the accuracy with which the experiments were made and presented. Careful work of this kind must be done by someone, in order that principles may be established definitely; and, ordinarily, such care is impossible. After these laws are determined, all that remains is to ascertain the practical uses to which the conclusions may be applied.

After reading the paper carefully and listening to Mr. Thompson's able abstract and Mr. Davis' timely discussion, the writer is fully convinced that the constructing engineer on any work of magnitude should know, with the materials which he has on hand, whether he is using the best, or nearly the best, proportions of the aggregates, in order to produce the best concrete. In many cases the stone will come from the same quarry, be passed through the same crusher and screens, and the sand will probably not vary to any great extent unless the locality from which it is obtained is changed. The engineer, therefore, should be in a position—if it can be done without the expenditure of too much time—to know what proportions of each of these given materials will give the best

Mr. Wagner.

results. Usually, he cannot afford the time to make a laboratory test, in a manner similar to that used by the authors, in order to see what kind of a curve his materials will give, but it would be a great satisfaction to him to see such a curve.

The authors used 21 sieves in their tests in order to determine accurately all points on their curves. It was undoubtedly wise to use this number in such experiments, but when applied practically it means that such a determination must be made in the laboratory. The writer would like to ask the authors whether it is not possible to reduce the number of these sieves to, say, four or five for the sand, and to a similar number for the stone? Could not these be taken into the field, with a weighing scale, so that an approximate curve might be determined there which would be of value?

If such a field outfit would enable an engineer to determine an approximate curve, it would not be a serious matter to go through the operation whenever the sizes of the sand or stone seemed to vary.

The engineer must determine from these results whether he is warranted in making any change which would entail additional cost, but, in most localities, the mere re-proportioning of the aggregate, without making any change in the proportion of the cement to the total aggregate, would entail no additional expense except the adjustment of the mixing boxes, and would, without doubt, make considerable difference in the character of the resulting concrete.

In recent years, the writer's attention has been called to the materials furnished on a number of works where the specified dividing line between the sand and the stone was $\frac{1}{4}$ in. In most of these cases the stone represents the "run of the crusher," and while not nominally containing any material smaller than $\frac{1}{4}$ in., actually had an amount of finer material. On the other hand, the sand contained some material which was larger than $\frac{1}{4}$ in., and yet the proportioning was made without any reference to the actual sizes. It seemed self-evident that there was too much so-called "sand," and not enough "stone," that is, there was an excess of fine material in the concrete.

The authors are asked whether they cannot suggest some satisfactory method to be used in the field which would enable the engineer on the ground to determine what changes, if any, should be made in the aggregates of the mix. Such a method, if practicable, would seem to have all the elements of scientific reasoning, and the small additional expense involved would be warranted on a work of any magnitude.

If, however, the results were to show that a re-crushing of the stone or a re-sieving of the sand were advisable, the question of the additional cost must govern the decision as to whether it would or would not be economical.

Mr. Feret. R. FERET,* ESQ. (by letter).—The writer has read this noteworthy paper with especial interest, as the methods followed by these investigators have been worked up in a spirit similar to that by which he himself has been animated; in fact, this study of concretes might be considered as a continuation of his own work on mortars. For example, he has already graphically represented the composition of granular mixtures, such as sand and gravel;† has instituted the calculation of the elementary volume and the density of mortars;‡ has insisted on the marked difference in the proportion of voids existing in the same mixture when dry and after gauging with water;§ and has shown that the volume of cement ought to be taken into account in the expression of the granulometric composition of the mixture, and that this latter is not modified by replacing a part of the cement with an equal absolute volume of sand of the same fineness, from which he draws the conclusion that the best granolithic composition is that which after gauging gives the densest mortar, the strength being governed by the proportion of fine grains as represented by the cement.‖

The writer is very much honored by the consideration which the authors have shown for his former works; as the basis of their researches rests precisely on the agreement in principles which have just been mentioned, he highly esteems the work of these investigators, and the remarks which he presents apply only to points of detail.

(a).—First, he believes that instead of using sieves with square meshes to determine the different sizes of grains, it would have been preferable to use metal sheets perforated with circular holes, these being simpler to define, less likely to be deformed, and more easily standardized. There exists, moreover (at least for the medium and large grains of sand), the simple relation, $x = \frac{3}{4}d$, between the diameter, d, of the holes in the sheet metal and the size of the square mesh of the sieve which, with any ordinary sand, passes the same proportion of residue.

(b).—From the fact that most concretes contain the same relative proportions of cement and aggregate, it results that the densest concretes contain, in an equal volume, more cement than the others, which explains in part their usual superiority in strength. To show in a better way that this superiority is due also to the density itself, it would have been preferable to have used in each such quantities of cement (calculated after the preliminary tests for density)

* Ancien élève de l'Ecole Polytechnique; Chef du Laboratoire des Ponts et Chaussées à Boulogne-sur-Mer.

† Commission des Méthodes d'Essai des Matériaux de Construction, Tome IV, p. 309.

‡ Sur la compacité des mortiers hydrauliques; *Annales des Ponts et Chaussées*, 1892, II, p. 50.

§ Chimie appliquée à l'Art de l'Ingénieur, 2d edition, p. 741.

‖ *Annales des Ponts et Chaussées*, 1892, II, pp. 57 et 157.

that the weight of the cement entering into the same volume of set concrete would have remained constant. Mr. Feret.

(c).—Concerning the difference in compacting which may result from more or less energetic ramming, it would have been well to determine the density on the blocks themselves before using them for strength tests. In reality, errors committed from the omission of this precaution are probably slight, for it seems that the concretes tested were all of a rather wet consistency. The effect of the ramming would be more important with dry concrete. On the other hand, it must not be forgotten that an increase in the proportion of water for gauging favors settling in lean concrete, so that laws resulting from the comparison of concretes mixed wet do not necessarily apply to those of dry consistency.

(d).—The transverse tests give a poor idea of the strength of the concrete, for the rupture depends chiefly on the composition of the concrete near the surface of the prism under tension, and, with a mixture having coarse ingredients, the distribution of these is far from being the same against the walls of the mould as in the interior of the mass.

(e).—As the authors have well illustrated in Table 7, the tests for compression on fragments of beams having their ends coated with neat cement show too great a strength. The experiments noted in Table 24, on cubes of mortar 71 mm. (2.8 in.) on a side, show that even a slight covering of mortar applied to the compressed faces of a block increases or diminishes its resistance according as the mortar is stronger or weaker than the block.

TABLE 24.

Description of specimens tested.	Average strength, in pounds per square inch.*
I.—Blocks Broken after Hardening Twenty Weeks.	
A. Blocks made wholly from mortar of test	1 250
B. Blocks in which the mortar had been replaced by a leaner mortar with a thickness of 0.119 in. on one of the compressed faces	1 210
C. Similar blocks with 0.590 in. of lean mortar	1 040
D. Similar blocks with 0.119 in. of neat cement	1 320
II.—Blocks of Another Mortar Tested at the End of Four Weeks.	
E. Blocks without caps	1 095
F. The same blocks, covered with a very thin coat of neat cement	1 250

* Each strength is the average of six tests.

In regard to the fragments of prisms tested by the authors, it is conceivable that the irregularity of their ends, notwithstanding the caps of neat cement, might have made the decidedly large differ-

Mr. Feret. ence between the strengths obtained, as appears from the averages found, 14.4% and 4.5%, respectively, for these variations. These difficulties might have been avoided and more exact results obtained by crushing the prisms transversely, placed cross-wise between two steel plates of the same size as the prisms themselves, the irregular ends extending from each side and exerting no influence on the strength of the cubic blocks actually submitted to the compression.

(f).—The writer does not think justifiable the distinction, made in Columns 23 and 24 of Table 8, between the voids of the cement and those of the aggregate, for the separation of the grains of cement in the mortar or in the concrete is not the same as in the neat cement paste. It would be more interesting to try to separate the total voids into "pores" of very small dimensions, produced by bubbles of air entrained in the mortar, or by the departure of the water in so far as it combines and evaporates, and "cavities," or relatively large voids, resulting from an incomplete filling of the spaces between the stones by the mortar. But the difficulty is to determine a limiting size between the grains which may be considered as forming part of the mortar and those which should be considered as stone.

(g).—The law discovered by the authors from which the maximum density of all concretes made with given materials corresponds to its granulometric composition defined by a certain curve, is extremely interesting. Unfortunately, it is necessary to determine for each kind of material the two parameters, a and b, which measure the axes of the ellipse, and, moreover, it seems to result from the reported experiments that the concretes corresponding to the theoretical curve often have too little cement to be used in practice. Finally, the authors' Conclusion 7, in connection with their statement that stones of uniform size give concrete at least as dense as stones of graduated sizes conforming to the rectilinear part of the theoretical curve, suggests that the actual formula is not definite enough, and should be replaced by another, more general and more simple. Particularly, it is desirable that new researches be undertaken, with the same care and the same earnestness, with a view to finding a law applicable to materials separated only into a limited number of sizes of grains.

(h).—Experiments upon permeability give in general uncertain results, and it is not unusual to see many blocks of the same concrete which, although treated in an identical manner, permit very different quantities of water to filter through them. Moreover, the flow varies very rapidly at first, each time that water at a higher pressure is applied. This law is not brought out clearly from the authors' tests, and Table 21 seems even to contradict it. It can be verified by the following experiment which the writer

Mr. Feret.

TABLE 25.—PERMEABILITY TESTS.

Age of blocks.				284	213	142	71	284	284
	Pressure of water, in pounds per square inch								
5 days....	Time of flow after commencement of test,* in minutes			20	28	36	45	60	180
	Block A_1†								
	Block B_1	Flow during 5 min., in grams		875	620	389	184	720	495
		Flow per minute, and per pound pressure	Each test	0.616	0.582	0.548	0.518	0.507	0.349
			Average			0.554			0.349
29 days....	Time of flow after commencement of test, in minutes			200	215	230	240	255	375
	Block B_1	Flow during 5 min., in grams		67	47	30	15	64	48
		Flow per minute, and per pound pressure	Each test	0.047	0.044	0.042	0.042	0.045	0.034
			Average			0.044			0.034
30 days....	Time of flow after commencement of test, in minutes			85	95	105	120	135	255
	Block A_2	Flow during 5 min., in grams		38	28	18	10	40	43
		Flow per minute, and per pound pressure	Each test	0.027	0.026	0.025	0.028	0.028	0.030
			Average			0.027			0.030
	Block B_2	Flow during 5 min., in grams		229	172	115	55	221	189
		Flow per minute, and per pound pressure	Each test	0.161	0.162	0.162	0.155	0.156	0.133
			Average			0.159			0.133
1 year....	Time of flow after commencement of test, in minutes			100	109	119	129	139	150
	Block A_3	Flow during 5 min., in grams		580	310	160	67	390	285
		Flow per minute, and per pound pressure	Each test	0.409	0.291	0.225	0.189	0.275	0.201
			Average			0.278			0.201
	Block B_3	Flow during 5 min., in grams		450	320	205	198	410	395
		Flow per minute, and per pound pressure	Each test	0.317	0.300	0.289	0.276	0.289	0.278
			Average			0.294			0.278

* In most of these tests the pressure of 284 lb. was reached at the end of about 15 min.
† At a pressure of 128 lb., water was forced through Block A, in large streams, and it was necessary to discontinue the tests.

Mr. Feret.

TABLE No. 26—PERMEABILITY TESTS.

Age of blocks.	Pressure of water, in pounds per square inch.....	284	213	142	71	284	412	284	General Average.
	Time of flow after commencement of test, in minutes.	120	131	141	151	165	180	190	
1 year.....	Block B_1. Flow during 5 min., in grams.....	108	77	49	24	95	158	97	
	Block B_1. Flow per minute per pound pressure..	0.076	0.072	0.069	0.068	0.067	0.077	0.068	0.071
	Block A_2. Flow during 5 min., in grams.....	177	140	100	59	158	234	162	
	Block A_2. Flow per minute per pound pressure...	0.125	0.132	0.141	0.166	0.111	0.114	0.114	0.129
	Block B_2. Flow during 5 min., in grams.....	165	119	77	38	153	260	157	
	Block B_2. Flow per minute per pound pressure...	0.116	0.112	0.108	0.107	0.108	0.126	0.111	0.113

has made, on two concretes, A and B, after various durations of storage in moist air, with each of which three identical sample blocks had been made. These samples are distinguished from one another in Table 25 by the indices. Moreover, Table 25 confirms the law indicated in the paper that the flow seems to be very nearly proportional to the pressure. Mr. Feret.

In all the experiments in Table 25 the pressure was maintained for a certain time at 20 kg. (284 lb. per sq. in.) between the next to last reading and the last, and the flow, almost always, was much diminished. In Table 26 it has been taken at an interval of 29 kg. (412 lb. per sq. in.), and then brought back to 20 kg. (284 lb. per sq. in.). It is noticeable then that the permeability increased as if the effect of the momentary increase of the pressure had been to open new passages for the water, or partly to clear out the passages already existing.

The principal conclusion to be drawn from the experiments in Table 26 is that in the quantity of water flowing through the same block during a unit of time there may be considerable variation, according to the conditions of time and pressure under which the previous filtration was carried on. At this place it may be proper to remark that, in Column 17 of Table 18, the authors have neglected to indicate the pressure maintained up to the first appearance of the filtering water.

The writer has recently been informed of experiments on the continued filtration of sea water through mortars, from which he concludes that it is often the case that the important initial diminution of permeability will be followed at the end of 8 or 10 days by a slight and temporary increase, and, with the method of conducting the experiments on the same subject,* which he has previously published, this might easily have passed unnoticed. He will be interested to know if some other investigator has not also observed this phenomenon.

W. F. Dennis, M. Am. Soc. C. E. (by letter).—The authors give the results of an elaborate and painstaking investigation in which they have sought a law for proportioning concretes. Mr. Dennis.

As the ultimate purpose of such investigations is to obtain results for practical application, the writer has examined the paper in order to find out how and to what extent the results would affect ordinary practice. The investigation proves that density and strength are fairly equivalent terms, and that the strength increases with the percentage of cement. These conclusions are the current understanding, and were to be expected.

In planning the investigation, the authors have had in mind that grading the aggregate into a number of different sizes, and

* *Annales des Ponts et Chaussées*, 1892, II, p. 97.

Mr. Dennis. the subsequent mixture of these graded sizes, would produce a concrete of maximum density and strength, and that an equation could be developed to show the best or an ideal proportion. This is shown by a sorting of the coarse aggregate, of a maximum diameter of 2¼ in., into fourteen different sizes of greater diameter than $\frac{2}{10}$ in.; and of a further sorting of the fine aggregate or sand into seven different grades of less than $\frac{2}{10}$ in. in diameter.

Quoting from the paper:

"The results as indicated in Table 13 show no marked difference between the specimens with graded coarse stone and those with uniform coarse stone. The former average slightly higher in strength, the latter have slightly greater density. * * * Expressed in another way, the effect upon the density and the strength is practically the same whether uniform stone or uniformly graded stone, or a mixture between these two is used."

In this it is understood that coarse stone is an aggregate in which the smallest stone is greater than one-tenth of the diameter of the largest stone.

Conclusions 1 and 2 state that a mixed aggregate, grading down from 2¼ in., gives a stronger and denser concrete than one grading down from 1 in., and that this, in turn, gives a denser concrete than one grading down from ½ in.; and in order to equalize the strength of the concrete made of aggregate grading down from 2¼ in., one-third more cement must be added in the case of the aggregate grading down from ½ in., and one-sixth more cement for the aggregate grading down from 1 in.

From these two general conclusions, the writer draws the following inference, which seems to be as valid as the conclusion drawn by the authors: That as no advantage in strength or density comes from using a small maximum aggregate, but the contrary, and that after the maximum diameter of aggregate is settled, both the strength and the density remain practically the same, whether the aggregate is mainly of the maximum size or of a mixture of that and smaller sizes, therefore, crusher-run stone of grades between 2¼ in. and $\frac{2}{10}$ in. will produce a concrete as good as that obtainable by any possible sorting of aggregates and any subsequent mixing in any graded proportions.

The importance of this conclusion is evident from the consideration that, on construction work, there is a grave disadvantage in having even two sizes of aggregate, because of the greater expense and detail. Any sorting to different sizes, and subsequent remixing, adds so much to the working cost that a theoretical advantage of mixing sized aggregates would soon be lost, even if the proof of that advantage was indisputable.

There remains the question, how are concrete materials to be proportioned in practice?

Mr. Dennis.

Cement certainly can be gotten into an agreed conventional equality between a weight and the space to be occupied by that weight, so that the unit of cement can be expressed either as a weight or as a volume.

The authors have assumed that 1 cu. ft. of Portland cement weighs 100 lb., corresponding to 3.8 cu. ft. to a commercial barrel, or 95 lb. to a commercial bag. The writer understands that this statement of relation is gaining in acceptance as a standard, and believes it to be a good standard.

Because the specific gravity of aggregates will vary greatly with the material selected, proportioning by weight will not produce equivalent mixtures, but will give higher or lower actual ratios of cement than a particular aggregate. Therefore the proportion should always be between the volume of cement and the volume of the various aggregates.

The uncertainty as to what any given mixture means, such as 1 : 3 : 5, for instance, comes from the uncertainty as to the cubic space to be accepted as the equivalent of a barrel of cement. It is on account of the convenience of proportioning by measurement that the writer believes it to be so desirable to fix, as has been done by the authors, an interchangeable standard of weight and volume for the cement, and, in calculations for mixture, he has always used a similar standard. The usefulness of an agreement upon this point is that the simple count of bags or barrels of cement is, in effect, a measurement of the cement; and that, with agreement upon the cubic contents of a bag of cement, the proportionate volumes of aggregates are fixed by simple multiplication.

In every-day work it is economically impracticable to use any more than one class of broken stone, gravel, or sand, and proportioning by volume measurement is the only practical way. It would follow that the relative proportions of cement, sand, and stone would have to be fixed either in the customary form of so many parts of each, meaning thereby volumes of each, or that the cement should be specified as a definite volume percentage of the total aggregate, leaving for experimental determination, during construction, the relative proportions between the quantity of large aggregate and the quantity of sand, leaving free the application of the authors' Conclusion 12:

"In practical construction use as small a proportion of sand and as large a proportion of stone as is possible without producing visible voids in the concrete."

Mr. Gaines.

RICHARD H. GAINES,* ESQ. (by letter).—The preliminary work, relating to the permeability of concrete and methods of waterproofing, undertaken at the Laboratory of the New York Board of

* Chemist, New York Board of Water Supply.

Mr. Gaines. Water Supply, embraced a comprehensive study of cement and concrete as a whole, including the materials, composition, testing, and proportioning, and experiments in the physical laboratory. Attention was then directed to the permeability of concrete, with a careful examination of the best known methods of water-proofing in present use. The methods hitherto devised may be roughly classified as follows:

1.—Processes involving the use of asphalt or other bituminous compounds, applied to the surface, directly or through any medium, the fundamental idea being to prevent the water from coming in contact with the concrete;

2.—Washes applied to the surface of the hardened concrete, the purpose being to fill the surface pores with some insoluble compound by precipitation;

3.—Substances added to the cement, or to the mortar at the time of mixing, hence forming an integral part of the concrete. The principle here is to fill the pores of the mass with some finely-divided insoluble material.

Owing to the fact that asphalt and the compounds belonging to this class are impervious to, and insoluble in, either salt or fresh water, methods in which these materials constitute the base have been used extensively for water-proofing for many years. A difficulty about these processes is that if such substances are put on directly, or through the medium of some bituminous paint as a binder, they can be applied only to dry surfaces, and therefore are not adapted to certain kinds of work. On the other hand, good authorities object to the use of felt of any kind in connection with underground water-proofing, especially about joints, owing to the practical impossibility of getting the felt tight. The greatest objection to the asphalt processes, however, is that the durability of the substances, for any considerable period, is still an open question. In a few years, at best, most of these bituminous substances oxidize, become brittle and porous, break down, and thus, after a time, cease to be effective for water-proofing.

Among the surface methods, the old Sylvester, or soap and alum, process, is fairly effective, but is very expensive for large work.

The examination of processes of the third class—in which materials are mixed with the cement—included absorptive, permeability, and strength tests, and analyses of the compounds. The results showed that some of the so-called "water-proofing compounds" were worse than useless. Put together without regard to chemical principles, they were found to consist in large part of inert substances, the particles of which, in cement mortar, acted as imper-

meable septa, interfering with normal setting, thus causing a decrease in tensile and compressive strength, while exerting a negative influence on permeability. Mr. Gaines.

Several of the processes examined were apparently based on sound principles, but were far too expensive to be considered seriously, except in special cases and for limited use.

The statement is made by one well-known authority on cement and concrete that additions of from 5 to 10% of hydrate of lime, replacing such proportions of cement at the time of mixing the mortar, will produce practical impermeability. This, however, is not in accord with the writer's experience, for his experiments in the Laboratory of the Board of Water Supply indicate that, while slaked lime slightly decreases the permeability, this advantage is more than offset by the decrease in the strength of the mortar. There is no chemical reason why other results should be expected.

Without going further into the merits or demerits of methods of water-proofing which have found more or less favor among engineers, it is sufficient to say that none of the processes examined gave results which would justify recommendation for extensive use.

Old Theories and New Principles.—In the study of the water-proofing problem, it was first necessary to determine, if possible, what actually happened in the wet mass. The accepted chemical theory is that the finely-ground, calcined mixture of carbonate of lime and clay, or argillaceous limestone of suitable composition, when mixed with water, solidifies and hardens, owing to the formation of hydrated aluminum calcium silicates, and basic calcium aluminates. During the chemical interchanges, calcium hydroxide is liberated, and, if the quantity of water present is limited, it soon becomes saturated with respect to this base, which crystallizes out, to be again replaced with more hydroxide, until the cement is hardened, silicates being left of a lower degree of basicity.

From a purely chemical viewpoint, the whole question of setting is still *in nubibus*, and the precise changes that take place will continue to offer a fruitful field for theory.

In the discussion of the decomposition of cement we are never dealing with homogeneous materials, but with a complex system—groups of bodies which undergo a plurality of chemical and physical changes, occurring either at the same time, or following each other, step by step. Such reactions can take place side by side, without mutual interference, just as sound, light, and electric waves, or as currents of heat, electricity, and diffusion, can pass through a medium simultaneously. The silica + lime + alumina + x in cement become, on the addition of water, an unstable system, which at once begins to rearrange itself. Several reversible reactions take place, involving dissociation, followed slowly by re-

Mr. Gaines. association. During this process of change and readjustment to a state of equilibrium, the unstable substances which are formed as intermediate products are not known separately, and their exact nature can only be conjectured.

Perhaps the meaning of the term "equilibrium" here should be made clear. Every system of bodies strives to approach a state in which the amount of energy which can be transformed into work (available energy) is as small as possible. When a chemical system reaches this state it is said to be in equilibrium. Every such state is really not to be regarded as a static but as a dynamic system, since the transformation of materials does not cease in a state of equilibrium, but the tendencies in both directions compensate one another, so that, as a result, no further alteration in the given chemical system can be detected.

After a careful survey of the unsatisfactory laboratory results obtained with the previously known processes of water-proofing concrete, the problem was attacked from a new standpoint. A pioneer attempt was undertaken to apply some of the theories and methods of physical chemistry to its solution.

Application of the Electrolytic Theory.—The amalgamation of the sciences of physics and chemistry during the last two decades has afforded the means of solving many complex problems. The ionic hypothesis, conceived by Faraday, and brilliantly developed by the great Swedish physicist, Arrhenius, connecting the facts of electrolytic changes with those of chemical interaction, marks a new era in our molecular conceptions, since it has revealed an entirely new species of molecule, *viz.*, the electrically-charged ion. With the help of this ingenious theory, the science of chemistry has undergone a renaissance, and many long-observed, but little-understood, phenomena have found explanation.

It has long been known (first observed by the Rogers brothers in the last century) that when certain finely-powdered minerals are treated with water, reactions take place which result in partial decomposition. Concerning this fact, investigation with the microscope has shown that water produces a colloid or glue-like surface condition of the particles, leading to plasticity in the wet mass, and increasing the binding power on induration. The last change is due to the surface of the particles becoming clogged with the product of hydrolytic decomposition. This general process is precisely similar to what follows when water is mixed with cement, but the reactions go farther in the latter case.

If we interpret the complex series of changes which take place in cement by the electrolytic dissociation theory and considerations developed in this study, the conclusion is reached that the following consecutive processes take place in the wet mass:

Mr. Gaines.

1.—Hydrolytic dissociation, *i. e.*, dissociation under the action of the ions of water;
2.—The formation about the particles of colloidal films which act as osmotic membranes, permeable to the ions of electrolytes;
3.—The absorption of basic ions by the hydrogels, resulting in coagulation;
4.—Recombination of dissociated molecules, tending to chemical equilibrium.

Functions of Electrolytes.—After some experiment and much study of the problems of water-proofing concrete, it was believed that, if the decomposition reactions could be made to go farther *i. e.*, with more reactivity in the mass, the hardened mortar would not only be rendered impervious, but, owing to the greater density, would have increased strength. The experiments of A. L. Cushman on the decomposition of feldspars give a valuable clue here. He found that treatment of powdered feldspars with dilute solutions of various salts (electrolytes) leads to an increased amount of decomposition products passing into solution. He obtained the same quantity of soluble decomposition products from a finely-ground feldspar by treatment with a dilute solution of ammonium chloride as could be extracted by electrolysis without subsequent regrinding. Cushman also found that the slowing up of hydrolysis upon finely-ground feldspar is due to the clogging effect of insoluble aluminum silicate which is precipitated in the colloidal form on the surface of the particles. He concluded that the effect produced by dilute solutions of salts was principally to set free the soluble products of hydrolysis absorbed by the colloidal decomposition products and thus expose fresh surfaces to the solvent.

The germ of truth thus laid bare suggested that the decomposition reactions in cement would be augmented in the same manner, namely: By replacing the mixing water with a very dilute solution of a suitable electrolyte—some substance the free charged ions of which would bring about a rearrangement of the molecules of the bodies with which they came in contact, causing a complete change in the equilibrium conditions of the system. The desired chemical and physical changes might be induced in part by catalysis, or accounted for either as electrical or chemical forces. Such actions, however, are usually regarded as merely manifestations of the affinities of one kind of matter for another.

We do not yet know the exact nature of chemical affinity. Berzelius long ago asserted the existence of a close connection between electrical and chemical forces. We know that chemical reactions produce electricity, and, conversely, that electricity often

Mr. Gaines. causes chemical action. The starting of a chemical change may come about through the agency of some very slight external influence in the form of electrical action. Clausius found that the smallest conceivable electric "force" is sufficient to effect the decomposition of an electrolyte. This involves the loosening of the chemical energy, since a part of the molecules must first split into atoms, or the union of atoms in the molecule must first be severed before the chemical reaction becomes possible. The reaction, once started, proceeds of itself.

These well-known facts are repeated in order to emphasize the intimate relation between chemical and electrical action, and, furthermore, the close relationship of matter and electricity. Indeed, the present-day idea among many physicists is that all homogeneous substances are synthesized from atoms which are collections of positive and negative charges held together mainly by their electrical attractions. Electricity is supposed to have an atomistic structure, with positive and negative elementary particles which are about thirty thousand times smaller than the atoms of ordinary matter. Just as free ions have an existence, so do free electrons have a momentary existence.

In our views of the phenomena of matter and electricity, we are apparently in a transition state between the atomic hypothesis and the electro-corpuscular theory, and these new lines of thought should not be lost sight of in any physical investigation.

In every chemical system there is a tendency to undergo change of some definite character. The conditions under which the system exists may be such that the affinities are in a state of stable equilibrium among themselves. Every system may be regarded as having a point of least resistance at which an alteration will most easily take place. If a suitable material were introduced into a chemical system it might react with certain constituents of the system to such an extent that what was before merely a tendency to undergo change would become an actual change beginning at the point which before the introduction of the material was the weakest point of the system.

Applying this theory to the decomposition of cement, it seemed probable that the electrolyte would form, with some constituent of the system, compounds too unstable to exist under the conditions, and that these compounds would immediately break up, leaving the ions of the electrolyte free to react with fresh portions of the system, a process repeating itself until a state of equilibrium was reached. The presence of the electrolyte, therefore, would cause a very different set of conditions from that existing in its absence —conditions which might easily bring the desired properties of impermeability along with increased strength to concrete. To

summarize, it was believed that the introduction of the electrolyte would cause this result: Mr. Gaines.

1.—By changing the equilibrium conditions of the system;
2.—By increasing the quantities of reacting substances in the mass—the fundamental idea of the "Law of Mass Action";
3.—By setting free the basic ions absorbed by the colloidal decomposition products, thus exposing increased surface to the action of the solvent;
4.—By catalysis, or electrical action set up in the mass by the excess of free ions, thus inducing chemical action.

Experiments, based on these conclusions, were made, and the results will be found in Table 27.

Colloids.—While pursuing the study of materials which might be adapted to rendering cement mortar water-proof, attention was soon attracted to the properties of certain colloidal substances. The important role played by the colloidal decomposition products of cement had already been observed and pondered over. There were reasons for believing that the process of setting was largely due to the coagulation of inorganic colloids, and the slowness of the action was in consequence of the indolence with which changes are prone to take place in these bodies. It seemed hardly possible that simple crystallization of calcic hydroxide or other compounds could give rise to the great strengths. These considerations led to a somewhat extended study of the properties of colloids, having in view a series of permeability and strength tests with additions of finely-ground colloidal clay in cement mortars. (The results of these experiments will be found in Table 27.)

Substances existing in solution fall into two groups, the crystalloids and the colloids, the physical and chemical behavior of which are characteristically different. Crystalloidal substances easily assume the crystalline condition, forming porous masses. The colloids are incapable of crystallizing, thus proving that the two classes of bodies are wholly different in molecular structure.

Many metallic oxides, and other inorganic compounds, such as are found in plastic clay for instance, have the power of entering into that peculiar hydrated, non-crystalline condition which Graham called "colloid." Later, Van Bemelen called the inorganic colloids "hydrogels." These curious substances consist of heterogeneous molecular complexes which possess a sub-microscopical, web-like formation, the distinguishing characteristic of which is the peculiar relation to water which they exhibit. Dried at moderately high temperatures up to a certain critical point, they will lose nearly all their water, only to take it back again eagerly when allowed to

TABLE 27.—EFFECT ON STRENGTH AND PERMEABILITY OF MORTAR, CAUSED BY REPLACING THE MIXING WATER WITH A DILUTE SOLUTION OF AN ELECTROLYTE; OR BY COLLOIDS IN THE MASS; OR BY BOTH.

Mortar.				Strength.						Permeability.				
				Compression.			Tension.			Grams of water passing in the last 10 min. of test.				
Cement, "Atlas." Parts.	Sand, "Cow Bay." Parts.	Fine Clay.	Alum Solution.	7 days.	28 days.	Specimens included in average.	7 days.	28 days.	Specimens included in average.	Pressure, 40 lb.	Age, in days.	Pressure, 80 lb.	Age, in days.	Specimen number.
1.—REPLACING THE MIXING WATER WITH A DILUTE SOLUTION OF AN ELECTROLYTE.														
1	3	.....	.....	1 180	1 635	3	155	205	6	527	28	524	28	1
										90	28	33	28	2
1	3	.....	2½%	1 105	2 010	3	206	229	6	62	28	10	30	1
										0	28	5	30	2
1	3	.....	5%	1 025	1 615	3	181	227	6	Trace.	29	2	30	1
										Trace.	29	2	31	2
2.—THE ADDITION TO THE MASS OF COLLOIDS IN THE FORM OF COLLOIDAL CLAY.														
1	3	.....	.....	1 180	1 635	3	155	205	6	527	28	524	28	1
										90	28	33	28	2
1	3 95%+	5%	.....	1 590	2 390	3	198	258	6	.....	.....	.....	.....	.....
1	3 90%+	10%	.....	1 895	2 350	3	270	335	6	0	30	0	32	1
										0	30	0	32	2
3.—REPLACING THE MIXING WATER WITH A DILUTE SOLUTION OF AN ELECTROLYTE, AND THE ADDITION OF COLLOIDS TO THE MASS.														
1	3	.....	.....	1 180	1 635	3	155	205	6	527	28	524	28	1
										90	28	33	28	2
1	3 95%+	5%	2½%	1 605	2 445	3	218	269	6	0	28	Trace.	30	1
										0	28	Trace.	30	2
1	3 90%+	10%	5%	1 855	2 945	3	262	342	6	0	28	Trace.	30	1
										0	28	Trace.	30	2

Mr. Gaines.

cool in free air or in moist atmospheres. This dehydration and rehydration can be repeated indefinitely unless the temperature of drying is carried too high, when the property is gradually lost and finally destroyed. A change in state impresses itself upon colloids, and determines the conditions in which these bodies will be ultimately found. In other words, a colloid seems to remember more or less perfectly a change it has suffered, just as does living matter. Mr. Gaines.

Many colloidal solutions are able to coagulate either on the addition of foreign substances or spontaneously. What the nature of the process is, and whether it is comparable to crystallization, cannot be stated. A coagulated solution presents several peculiarities —an intermediate thing between a solid and a liquid, its enormous internal friction relates it to solids. The simplest conception of a hydrogel is that a solid substance is separated which has a "network" structure, the interstices of which are filled with water which is held fast by capillary forces.

The properties of substances which assume the colloidal state are in part due to the approach of each particle to its molecular state with a resulting proportional increase of active surface. If we imagine certain inorganic substances divided under water into smaller and smaller particles until this dust is able by virtue of its minuteness to remain suspended in the liquid an indefinite length of time, then we have a colloidal solution. Such a solution represents a suspension of fine particles, electrically-charged, from giving off ions, just as have electrodes. If now, the suspended particles become steadily smaller and smaller while at the same time their electric charge grows, they approximate more and more the behavior of ions, until finally the colloid passes over into the crystalloid, which dissociates in aqueous solution into its strongly charged ions. It can easily be seen how, through changes in only a few conditions, an enormous variety in the character of the colloidal reactions is brought about, a behavior apparent in numerous transitions between colloids and crystalloids.

Other physical criteria of colloidal solution are: Sensitiveness to an electrolyte, and movement in an electric field. Colloids, when placed in a field of force, migrate, some to the cathode and some to the anode. In this respect they behave as electrically charged particles. Thus ferric hydroxide is a positive colloid and moves to the cathode, and silicic acid is a negative colloid and moves to the anode. Electrolytes cause the precipitation of colloidally dissolved substances, and the degree in which this property is manifested depends on the electric charge of the ion rather than on the chemical nature of it.

Not only can electrolytes cause the precipitation of colloids, but one colloid can also cause the precipitation of another. In this

Mr. Gaines. case there is a certain optimum concentration of the colloid. When a colloidal precipitate is formed in the presence of an electrolyte, a portion of the cations (basic ions) of the electrolyte is absorbed by, or adsorbed on, the colloidal matter, in the form of the hydroxide, while the anions (acid ions) remain in solution. Cushman has shown that in certain cases the base will be absorbed from solutions of electrolytes by colloid substances, the acid ion remaining free in solution. In other cases the acid will be taken and the base left, and again the whole salt will be occluded. This absorption power is in no sense due to chemical combination, but seems to depend upon a sub-microscopic physical property of the colloid. Furthermore, there is evidence which points to the cause of the phenomenon as due to electro-static conditions maintained between the ions of the electrolytes and the surface charges of the particles.

The colloids are an excellent means, under certain circumstances, of keeping slightly soluble salts in solution, in that they themselves unite with the salt ions and form molecular aggregates without necessarily being precipitated. In the setting of cement, this is a most useful function of colloids. By this means the precipitation of the scarcely soluble salts is retarded or suppressed so that the action can take place in the time desired, and with a gradual evolution of heat. The function is similar to that performed by a negative catalyser, and is of vital importance, remembering that the amount of available energy in the form of heat produced in a chemical reaction is always the same, whether the reaction does or does not take place slowly.

The hardening of cement is probably not so much a chemical as a physical process, and may not be due to the formation of chemical compounds at all, but to a specific interaction of colloids and electrolytes which protects the calcium compounds from quick hydration, with consequent sudden increase in volume, due to rise in temperature. The deduction follows that, without a properly balanced mixture of colloids and electrolytes in the mass, a theoretically perfect cement, in strength and impermeability, would be impossible.

In all these considerations a proper amount of mixing water for optimum setting conditions is, of course, assumed. It must be remembered that a substance in solution behaves as a gas, occupying the same volume as the solution, and the laws which substances in solution obey are analogous to those which are obeyed by gases (Van't Hoff's law). Hence, an excess of water weakens cement, because the spaces between the nidi, or the particles of the active mass, become too great for uniform coherency in the hardened product.

These studies suggested a series of permeability and strength

tests on cement mortars with additions of different quantities of a suitable electrolyte and some form of colloidal matter, or both. The electrolyte chosen was ordinary alum, dilute solutions of which were used to replace the mixing water ($2\frac{1}{2}\%$ and 5% solutions). The form of colloid selected was that contained in specimens of finely-ground clay. Many clays contain a large amount of substances which assume the colloidal state on the addition of water, which sets free gelatinous decomposition products by hydrolysis. Mr. Gaines.

For the results obtained on permeability recorded in Table 27, 2-in. cubes were used; for tensile strength, ordinary standard briquettes; and for compressive tests, 2-in. cubes.

Summary.—The results from this series of experiments indicate that water-tight concrete may be obtained in three ways:

1.—By replacing the mixing water with a dilute solution of a suitable electrolyte. The concentration of the solution need not exceed 2% in strength. A 1% solution is believed to be sufficient, since greater concentrations are not further ionized.

2.—By substituting for from 5 to 10% of the cement an equal quantity of dried and finely-ground colloidal clay, intimately mixed with the cement.

3.—By the use of both the solution of the electrolyte and the clay.

Table 27 shows that these substitutions not only give a water-tight product, but, for the periods of 7 and 28 days, give greatly increased strengths.

This study was conducted in the laboratory established by the Board of Water Supply, of which J. Waldo Smith, M. Am. Soc. C. E., is Chief Engineer, the laboratory being attached to the Headquarters Department, of which Alfred D. Flinn, M. Am. Soc. C. E., is Department Engineer.

Messrs. Fuller and Thompson.

WILLIAM B. FULLER AND SANFORD E. THOMPSON, MEMBERS, AM. Soc. C. E. (by letter).—Mr. Davis' discussion brings out very clearly the application of the methods described in the paper to a number of other aggregates. Fig. 18, illustrating the run of the bank and how it can be materially improved by screening into two sizes and re-combining, is particularly applicable, because it shows that it is better to separate some materials into three screenings before combining them. Figs. 20 and 21 are also interesting in this connection. Mr. Davis does not make it quite clear in his discussion that, in making up any given proportions of concrete, the proportion of fine material also includes the cement. Taking the material of Fig. 18, for example, if 1 : 9 concrete is desired, the mixture by weight will be 10% cement, 24% fine material, and 66% coarse material.

Messrs. Fuller and Thompson.

Fig. 20 illustrates what is, perhaps, the most frequent fault to be found in concrete mixtures, the presence of too much medium-sized stone in the aggregate.

In reply to Mr. Parsons, no tensile tests of concrete were made. These tests were considered, and correspondence was had with the authorities of Columbia University with reference to a proper shape of specimen, but the time was too limited to undertake the tests. The results of the beam tests give records of ultimate transverse strength from which safe working values may be estimated for similar materials.

In regard to age, it may be stated that concrete increases rapidly in strength up to about three months, after which the increase is much slower. The effect of time was eliminated purposely, as far as possible, by testing all the beams at nearly the same period. There is no reason to assume that grading the aggregate would have an appreciable effect upon growth in strength. Recent tests made in Chicago indicate that a dense concrete resists heat better than a poorly proportioned concrete.

Mr. Hazen, in his analysis of Mr. Davis' diagrams, fails to take into account that, in proportioning concrete of any given strength by the authors' method, a given proportion of the fine material, as shown on the diagram, is replaced by the same weight of cement; and it would appear, therefore, that in the examples cited by Mr. Davis, an aggregate containing very much less than one-third of sand gives the best results. Taking Fig. 21 representing 1 : 6 concrete, we have the proportions, 14 : 20 : 10 : 56, or changed into ordinary nomenclature, 1 cement, 1.4 fine, 0.7 medium, 3.9 coarse.

It is true, of course, as Mr. Hazen suggests, that, in practice, in a great many cases, special grading of the aggregate is not warranted, and the authors have no desire to advocate such grading under all conditions. They do believe, however, that no matter how small the job, it is not a very difficult matter to procure three or four small sieves and a grocer's scales and make an analysis of the material as used. The plotting of even one such analysis curve will indicate the direction in which improvements can be made, and very often much improvement is possible without additional cost, not infrequently, in fact, at a saving in cost.

On the other hand, there is a demand for economy in laying concrete by more accurate proportioning; and it is not over-stating the fact to say that in the use of no other structural material is so much money thrown away because of unscientific practice. The materials at Jerome Park were all good in themselves, and ordinary proportions gave excellent results, yet even with the saving suggested by Mr. Hazen—25 cents per cu. yd.—there would be a daily reduction of $125 in cost on a work requiring the laying of

Messrs. Fuller and Thompson.

500 cu. yd. per day, less the additional cost of separation into one or two more sizes, which, at a maximum, would not have amounted to more than $45 per day. On the other hand, with many materials there would have been a very much greater difference. In one case, for example, where tests were made for special grading for concrete blocks, it was found that, by increasing slightly the maximum size of the aggregate and separating it into three sizes instead of using it as it came from the crushers, a mixture with proportions of 1 : 7 could be used, which was as strong and dense as had been made with proportions of 1 : 4.

Of course at Jerome Park the large number of sieves were selected with a view to arriving at laws and principles; in practice a less number would be required, as Mr. Wagner suggests. Six or seven sieves for the sand and three or four for the stone are sufficient for the ordinary laboratory. In the field it is obviously impossible to separate the material into very fine sizes, but in many cases it is practicable and economical to use three or four sizes of aggregate, instead of two.

The authors are honored by Mr. Feret's very interesting discussion, and they agree with him in many of the points of detail which he suggests.

(a).—Screens with round holes undoubtedly give a more uniform product than wire mesh, but it was impossible to obtain them in the smaller sizes.

(b).—While previous fixing of the proportions, so that in each case a given weight of cement would enter into the same volume of concrete, would have shown better comparisons, the authors felt that, with so many compositions of aggregates, it was impossible to reach results in the preliminary tests of density which would compare exactly with the beam tests and, if they did not compare, another variable would have been introduced.

(c).—The density of the beams was determined by weighing all the materials and by measuring the specimens.

(d).—Compression tests are better than transverse tests, but a compression machine was not available when the tests were outlined, and the specimens which were tested for compression had to be sent to Columbia University. The variation in the results is unquestionably due, in part, to the use of transverse tests, and if a second series had been made undoubtedly the authors would have selected compression tests even at a great inconvenience.

(e).—The compression specimens were tested on end so as to have longer prisms. In so doing, however, as Mr. Feret suggests, another source of error was introduced, which was probably more serious than the shape of the specimen.

Mr. Dennis draws the conclusion that, because the ideal curve

Messrs. Fuller and Thompson.

suggests a straight line for the aggregate above 0.2 in., the run of the crusher, therefore, is preferable. This conclusion is correct provided the product of the crusher or the gravel bank gives an evenly graded aggregate, that is one whose mechanical analysis plots as a straight line; unfortunately, however, all stones do not crush or run naturally in this manner, and often there is a large excess of medium stone. One of the great advantages of mechanical analysis is to be able to select a stone which will give an evenly graded aggregate.

Mr. Gaines suggests a new theory for the water-tightness of concrete which is of much interest and worthy of further investigation and tests.

Charles F. Marsh, M. Am. Soc. C. E., has kindly called the attention of the authors to an error and also to a lack of clearness in the discussion of the formulas for the ellipse as published in *Proceedings,* and these are corrected in the present issue.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 1054.

A SOLUTION OF THE PROBLEM OF DETERMINING THE ECONOMIC SIZE OF PIPE FOR HIGH-PRESSURE WATER-POWER INSTALLATION.*

By Arthur L. Adams, M. Am. Soc. C. E.

With Discussion by Messrs. W. L. Butcher, W. E. Buck, P. E. Stevens, C. L. DeMott, E. Kuichling, William J. Roberts, Charles D. Marx and C. B. Wing, James H. Wise, E. W. Myers, Asahel C. Toll, F. G. Baum, and Arthur L. Adams.

In discussing a paper† by Edwin H. Warner, M. Am. Soc. C. E., descriptive of a large water-power installation, the writer called attention to the interesting problem of design, in such installations, involved in the determination of the most economic size of pressure pipe.

Between the pipe of infinite diameter which would be necessary to utilize the entire static pressure, and a pipe of diameter so small that the entire head would be consumed in initial velocity at entrance and in overcoming resistances, there is, of course, some point representing the most economic adjustment.

At that time, the writer, reasoning from analogy, suggested that Sir William Thomson's law for electric transmission, requiring

*Presented at the meeting of June 5th, 1907.

†"The Hydraulic Plant of the Puget Sound Power Company," *Transactions*, Am. Soc. C. E., Vol. LV, p. 228.

that "the relation between the investment in copper and the cost of the power consumed in line losses should be such that the annual interest on capital outlay should equal the annual cost of the energy wasted," was also applicable to the transmission of power through the medium of water flowing in pressure pipe.

More deliberate consideration of the subject has convinced the writer that this conclusion was wrong. He is able now, however, not only to present what he feels confident is the correct solution; but also, as a result, to state a very simple rule applying to all such cases with sufficient exactness, which he hopes and believes will be of substantial value to the profession.

This solution is based upon the following proposition, which is self-evident: That the relation between the investment in any pipe line and the value of the energy lost in frictional resistance to flow is most economical wherein the cost of the pipe plus the value of the energy sacrificed is a minimum. Or, if one chooses to state it in terms of the annual cost: Wherein the annual cost of the pipe line plus the value of the energy annually sacrificed is a minimum. The writer has worked out the solution on the basis of the first manner of stating, but, as will be seen, the rule is equally applicable to either proposition. Table 1 is self-explanatory, and is made the basis of the graphical solution hereinafter referred to.

In compiling Table 1, the frictional loss is computed by the Chezy formula, $S = \left(\frac{V}{C\sqrt{R}}\right)^2$, in which V is the velocity, in feet per second; R is the hydraulic radius (one-fourth the diameter); C is a coefficient, taken in this table uniformly at 100; and S is the frictional loss, in feet per linear foot of pipe. It will be observed that the frictional coefficient, C, is taken as a constant. For riveted pipes, the type generally used, this is fully warranted by the results of experiment, while, for other classes of conduit, which have been in use for a considerable length of time, such an assumption will lead to no material error.

Two other assumptions are deserving of mention: First, no account is taken of the additional thickness necessary at the upper end of the pipe line irrespective of the requirements to resist pressure, an item of negligible importance in the solution of this problem; and second, laps, rivets and overweights of sheets are not

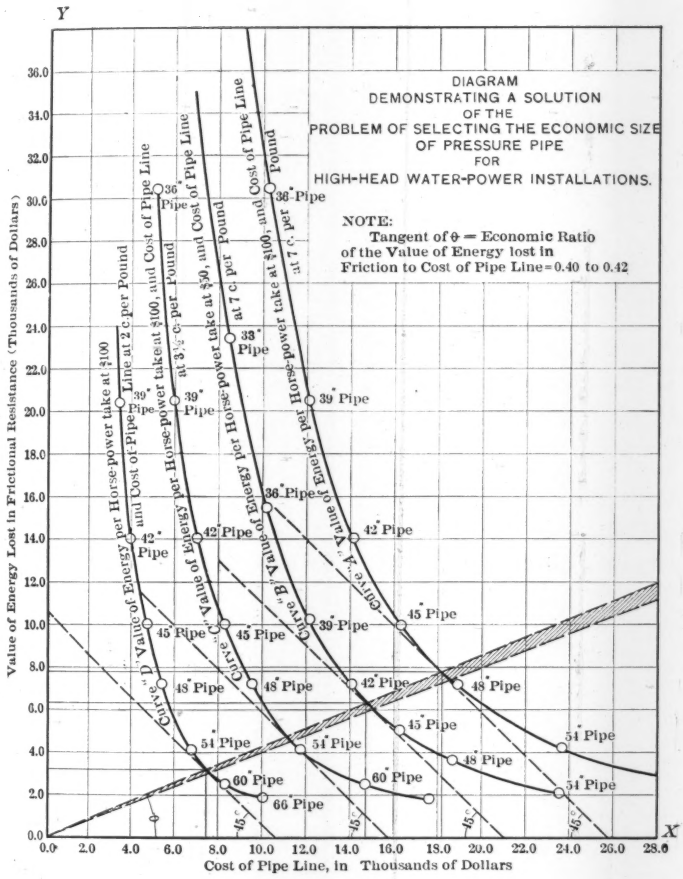

Y
X
DIAGRAM
DEMONSTRATING A SOLUTION
OF THE
PROBLEM OF SELECTING THE ECONOMIC SIZE
OF PRESSURE PIPE
FOR
HIGH-HEAD WATER-POWER INSTALLATIONS.
NOTE:
Tangent of θ = Economic Ratio
of the Value of Energy lost in
Friction to Cost of Pipe Line = 0.40 to 0.42
Value of Energy Lost in Frictional Resistance (Thousands of Dollars)
Cost of Pipe Line, in Thousands of Dollars
Curve "D" Value of Energy per Horse-power take at $100 Line at 2 c. per Pound and Cost of Pipe Line
Curve "C" Value of Energy per Horse-power take at $100, and Cost of Pipe Line at 3½ c. per Pound
Curve "B" Value of Energy per Horse-power take at $100, and Cost of Pipe Line at 5 c. per Pound
Curve "A" Value of Energy per Horse-power take at $100, and Cost of Pipe Line at 7 c. per Pound
θ
45°

FIG. 1.

included in computing the weight of the pipes, as these may be regarded as bearing a constant relation to the weight of the pipe, and consequently will not affect the rule sought. The weights of pipes for given pressures, therefore, are taken as proportional to the squares of their diameters, an assumption which is substantially correct.

TABLE 1.—Comparative Data for the Study of the most Economic Diameter of Pipe to be Installed for Conducting 100 Cu. Ft. of Water per Second from a Head-Race to the Water-Wheels; the Pipe being assumed as Vertical, and 1 000 Ft. in Length.

(1) Diameter of pipe, in inches.	(2) Friction loss, in feet in 500 ft. (= ¼ total length).	(3) Average dynamic pressure, in feet.	(4) Average strain in pipe shell, in pounds per linear inch.	(5) Average thickness of pipe, in inches, at a strain of 10 000 lb. per sq. in.	(6) Weight of pipe, in pounds per foot, without allowance for seams or overweight.	(7) Cost of pipe per foot, at 7 cents per lb.	(8) Value of energy lost in friction in 1 000 ft., at $100 per horse-power.	(9) Value of energy lost in friction in 1 000 ft., at $50 per horse-power.
30	33.3	466.7	3 024	0.302	97.5	6.83	75 440	37 720
33	20.6	479.4	3 417	0.342	121.5	8.50	46 683	23 342
36	13.4	486.6	3 783	0.378	146.5	10.26	30 357	15 178
39	9.0	491.0	4 136	0.414	173.8	12.17	20 389	10 195
42	6.2	493.8	4 480	0.448	202.8	14.20	14 047	7 023
45	4.4	495.6	4 817	0.482	233.5	16.35	9 968	4 984
48	3.2	496.8	5 151	0.515	271.6	19.01	7 249	3 624
54	1.8	498.2	5 811	0.581	338.3	23.68	4 078	2 039
60	1.1	498.9	6 465	0.647	418.4	29.29	2 492	1 246
66	0.8	499.2	7 116	0.712	506.6	35.46	1 812	906

The diagram, Fig. 1, presents four curves, designated as *A*, *B*, *C*, and *D*, covering a wide range of assumptions as to the cost per pound of the pipe line in place and the value or cost of the energy sacrificed in frictional resistance. Two of these, *A* and *B*, are plotted from Columns 8 and 9 of Table 1. The figures for *C* and *D* are also readily deduced from the data in Table 1.

A common scale is used for the axes of X and Y. Those points on the curves which satisfy the requirements of $X + Y =$ a minimum will be found in the diagram to coincide with the points of tangency with right lines drawn at an angle of 45° with both

ordinates, because the values of $X + Y$ increase in either direction therefrom.

Now, it will be observed that these points of tangency for all four curves lie on a common straight line through the origin, from which it follows that the tangent of the angle, θ, expresses the economic ratio between the value of the energy sacrificed in friction and the cost of the pipe line; and this ratio is a constant whatever may be the unit values of either of these functions. Graphically, it is not possible to ascertain the mathematically exact value of tan. θ; practically, it is not necessary. It is from 0.40 to 0.42, and for convenience of memory may be stated as the former.

Since, in financial management, annual cost bears a constant percentage relation to capitalized cost, the value of tan. θ will be unchanged, as stated at the beginning, by using either the one or the other.

The writer, therefore, would submit as the rule defining the true economic relation to be observed in designing pipes for high-pressure water-power installations the following, the volume of discharge, of course, being fixed:

That pipe fulfills the requirements of greatest economy wherein the value of the energy annually lost in frictional resistance equals four-tenths (0.4) of the annual cost of the pipe line.

The writer regrets that, having long since lost possession of his calculus, he finds himself unable to verify the rule mathematically, but he hopes someone able to do so will feel interest enough in the subject to undertake it.

DISCUSSION.

Mr. Butcher.

W. L. BUTCHER, ASSOC. M. AM. SOC. C. E. (by letter).—The problem presented here is similar to the one encountered in the design of water and sewage disposal works, as to the most economical size of force main, for which the writer has worked out a formula.*

In regard to the problem under consideration, using the author's assumptions as to cost and perpendicularity of pipe, value of C, etc., a formula may be deduced, as follows:

Let x = the diameter of the pipe, in feet;
a = the head, in feet, on the section under consideration;
b = the cost of the pipe, in dollars per pound;
d = the value of the energy of 1 cu. ft. per sec. with a head of 1 ft.;
f = the total loss of friction in the above section;
Q = the quantity flowing, in cubic feet per second;
10 000 = the tensile stress in the steel, in pounds per square inch;
490 = the weight of 1 cu. ft. of steel, in pounds;

then, the cost of a section of pipe 1 ft. long, under a head $a = \left\{\frac{\frac{62.4\ (a-f)\ x}{2}}{1\ 440\ 000}\right\} 490\ \pi\ x\ b$ (thickness of pipe shell neglected in calculating circumference) $= 0.033\ a\ b\ x^2 - 0.033\ f\ b\ x^2$;

also,
$$V = c\sqrt{r\ s},$$

or
$$\frac{\frac{Q}{\pi\ x^2}}{4} = c\sqrt{\frac{x}{4}\ s},$$

or
$$s = \frac{0.00065\ Q^2}{x^5}.$$

Whence the value of the energy lost in passing 1 ft. of pipe $= \frac{0.00065\ Q^3\ d}{x^5}$.

For the most economical diameter of pipe, the cost of the pipe plus the value of the energy lost must be a minimum, or, in the terms of the calculus, the first differential of $0.033\ a\ b\ x^2 - 0.033\ f\ b\ x^2 + \frac{0.00065\ Q^3\ d}{x^5}$ should be placed $= 0$; that is,

$$0.066\ a\ b\ x - 0.066\ f\ b\ x - \frac{0.00325\ Q^3\ d}{x^6} = 0;$$

or,
$$x = \sqrt[7]{\frac{0.0492\ Q^3\ d}{a\ b - f\ b}}.$$

* *The Engineering Record*, May 13th, 1905.

By this equation the proper diameter at any point may be determined. In solving this equation, a value of f must be assumed, and afterward corrected, if necessary. However, considering the limitations of the formula, the value of f can usually be approximated near enough at the first trial. Mr. Butcher.

TABLE 2.—EXAMPLE OF THE APPLICATION OF THE FORMULA TO AN ACTUAL CASE, WHEN THE VALUE OF ONE HORSE-POWER = $100; COST OF PIPE = $0.07 PER POUND; $Q = 100$ CU. FT. PER SEC.; AND HEAD = 1 000 FT.

Section.	Diameter, in feet.	Thickness of shell, in inches.	Weight of pipe, in pounds.	Cost.	Velocity, in feet.	Friction loss, in feet. $C = 100$.
0 to 200	4.5	$\frac{1}{4}$	29 000	$2 030	6.3	0.70
200 " 400	4.1	$\frac{5}{16}$	46 000	3 220	7.5	1.10
400 " 600	3.9	$\frac{3}{8}$	62 000	4 340	8.5	1.48
600 " 800	3.7	$\frac{11}{16}$	78 000	5 460	9.4	1.91
800 " 1 000	3.6	$\frac{15}{16}$	86 000	6 020	9.7	2.09
Totals....	.....	.....	.....	$21 070	.....	7.28

The expression,

$$x = \sqrt[7]{\frac{0.0492\ Q^3\ d}{a\ b - f\ b}}, \text{ may be written as } a\ b\ x^2 - f\ b\ x^2 = \frac{0.0492\ Q^3\ d}{x^5};$$

or $$2\ (0.033\ a\ b\ x^2 - 0.033\ f\ b\ x^2) = \frac{5\ (0.00065\ Q^3\ d)}{x^5};$$

that is $$2 \times \text{cost of pipe} = 5 \times \text{value of energy lost},$$

which is the same result as obtained by the author's graphical method.

With a head of 7.28 ft., 100 cu. ft. per sec. means a loss of 82.7 h-p., having a value of $8 270, or practically four-tenths of the cost of the pipe. Although in Table 2 it is assumed that the pipe is vertical, it will be observed that such an assumption is not necessary in order to apply the formula.

W. E. BUCK, M. AM. SOC. C. E. (by letter).—To develop a formula for this purpose, in addition to the nomenclature used by the author, Mr. Buck.

let Q = the discharge, in cubic feet per second;
L = the length of the pipe line, in feet;
D = the cost of the pipe line;
d = the diameter of the pipe, in inches;
b = the value of 1 h-p.;
B = the value of the power lost;
m, k, k', and f are constants, as explained in what follows.

Mr. Buck.

The cross-section of the pipe, in square feet $= \frac{\pi\, d^2}{576}$.

Hence $V = \frac{576\ Q}{\pi\ d^2}$.

$R = \frac{1}{4}$ the diameter, in feet, $= \frac{d}{48}$.

$$S = \frac{576^2\ Q^2}{C^2\ \pi^2\ d^4 \times \frac{d}{48}} = \frac{1\,613\,565\ Q^2}{C^2\ d^5}.$$

The total loss of head $= S\ L$; the loss of horse-power $= 0.1134\ Q\ S\ L$, and the value of the power lost $= 0.1134\ b\ Q\ S\ L$. Substitute the value of S, and $B = \frac{182\,978\ Q^3\ L\ b}{C^2\ d^5}$. Let $m = \frac{182\,978\ Q^3\ L\ b}{C^2}$ and $B = \frac{m}{d^5}$.

If the cost of the pipe line varies as the square of the diameter, $D = k\ d^2$; $B + D = \frac{m}{d^5} + k\ d^2$, and this is to be a minimum. Equate the first differential of this to zero, and $-\frac{5\ m}{d^6} + 2\ k\ d = 0$; from which $d = \left(\frac{5\ m}{2\ k}\right)^{\frac{1}{7}}$. Substitute this value in the expressions for B and D, and $\frac{B}{D} = \frac{m}{\left(\frac{5\ m}{2\ k}\right)^{\frac{5}{7}}} \div k\left(\frac{5\ m}{2\ k}\right)^{\frac{2}{7}}$; which reduces to 0.4, the value given by the author.

Applying the formula, $d = \left(\frac{5\ m}{2\ k}\right)^{\frac{1}{7}}$, to the conditions represented in the diagram, Fig. 1, the results are as follows:

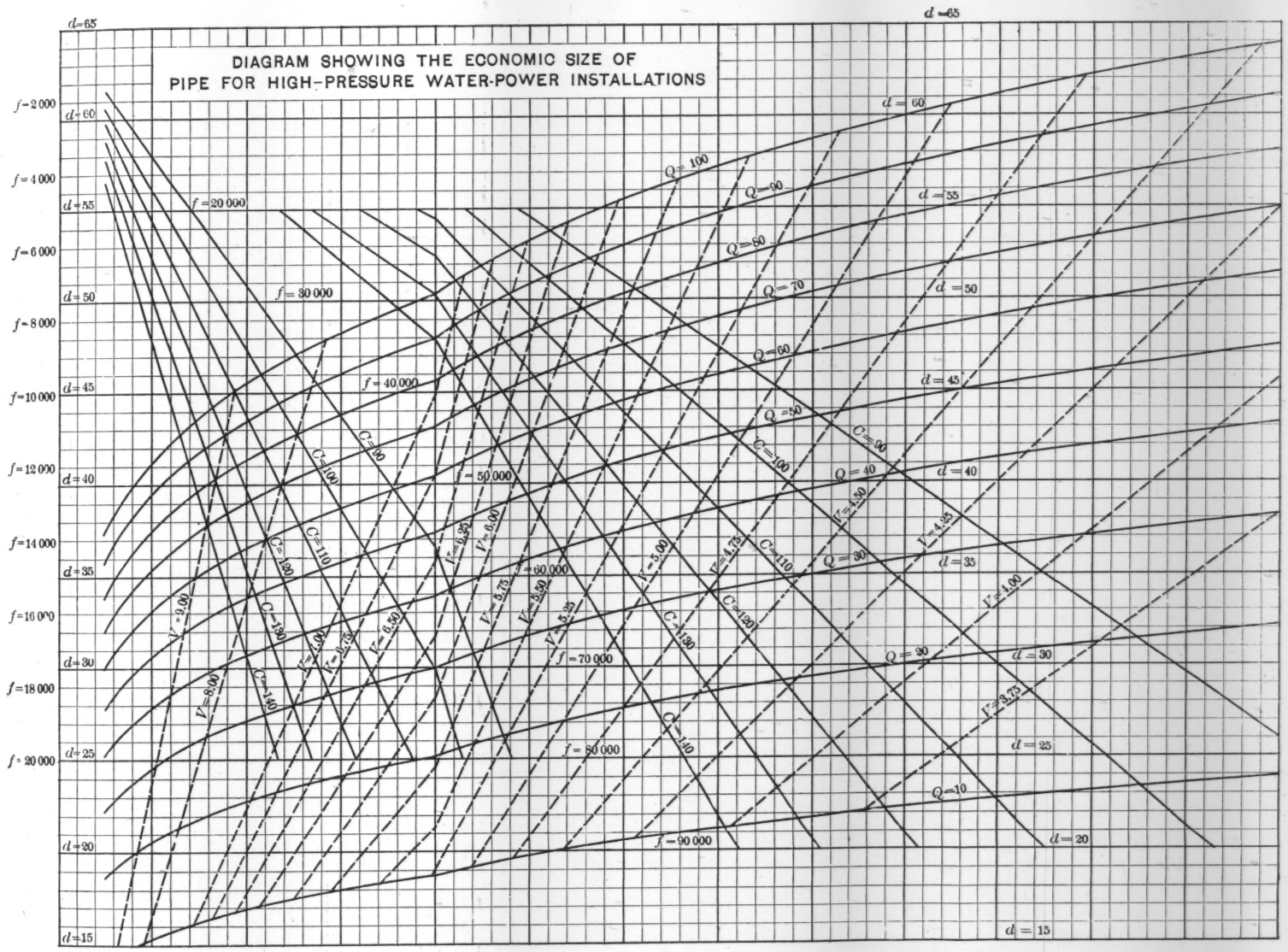
For the conditions of Curve "A", $d = 47.78$
" " " "B", $d = 43.27$
" " " "C", $d = 52.75$
" " " "D", $d = 57.14$

These values agree with the diagram.

It should be noted that the values given by the author in Columns 8 and 9 of Table 1 do not follow exactly the formula given above, but they should, if the computations for loss of head were carried out sufficiently.

The cost of pipe per foot, as given in Column 7, does not vary exactly as the square of the diameter, because the author has proportioned the pipe to the dynamic head instead of the static head.

PLATE XV.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LIX, No. 1054.
BUCK ON
SIZE OF WATER POWER PIPE.



Mr. Buck.

There would seem to be no occasion for this, as the more the dynamic head falls below the static head the greater the danger from water hammer.

In the formula given above, L is a factor in the value of m, and also in the value of k. If $k = L\ k'$, so that $k' =$ the cost per foot of pipe divided by the square of the diameter, $\frac{5\ m}{2\ k}$ reduces to $\frac{457\ 445\ b\ Q^3}{C^2\ k'}$, and $d = \left(\frac{457\ 445\ b\ Q^3}{C^2\ k'}\right)^{\frac{1}{7}}$. Let $\frac{b}{k'} = f$, and $d = \left(\frac{457\ 445\ f\ Q^3}{C^2}\right)^{\frac{1}{7}}$.

The diagram, Plate XV, has been constructed from this formula, and is used in the following manner: Enter the diagram at the value of f, proceed horizontally to the right, to the line representing the proper value of C, then proceed vertically to the line representing Q, and the value of d is shown by the horizontal lines. For values of f ranging from 2 000 to 20 000, the C lines at the left must be used; and for values of f ranging from 20 000 to 90 000, those at the right must be used. The dotted lines show the velocities, in feet per second.

The diagram may be used to design compound pipes. For example, suppose a pipe line to deliver 70 cu. ft. per sec., under 800 ft. head, and that the pipe is to be built in four sizes; and, further, assume $b = 110$ and $C = 120$.

By preliminary estimate, it is found that the average cost per foot of a 40-in. pipe will be:

For heads	0 to 200 ft.	$2.90,	for	which	$f = 60\ 700$
"	"	200 to 400 "	5.70	"	"	$f = 30\ 900$
"	"	400 to 600 "	9.75	"	"	$f = 18\ 050$
"	"	600 to 800 "	13.80	"	"	$f = 12\ 750.$

Then, by the diagram, Plate XV:

For head	0 to 200 ft.,	$d = 48.8$	and	$V = 5.4$
" "	200 to 400 "	$d = 44.4$	"	$V = 6.5$
" "	400 to 600 "	$d = 41.0$	"	$V = 7.7$
" "	600 to 800 "	$d = 39.0$	"	$V = 8.4$

The formula and the diagram are both based on the total theoretical power lost, and the value of b to be used should be the actual value of a horse-power at the wheels multiplied by the percentage of efficiency of the wheels.

The break in the curves of the diagram at the fourth heavy vertical line is occasioned by a change of horizontal scale to make the portion at the left of this vertical more easily used.

Mr. Stevens.

P. E. STEVENS, ASSOC. M. AM. SOC. C. E. (by letter).—The formula derived by Mr. Adams should prove very useful in this field, provided its limitations are kept clearly in mind. It is not sufficient to say that a design is the most economical, without stating the nature of the economy effected.

In planning to develop water-power from a given stream it is first necessary to ascertain what the proprietor of the rights wishes to derive therefrom in the way of benefits. It may be required to develop a given net power—within the capacity of the stream—at a minimum first cost or at a minimum annual cost; it may be required to develop a maximum of power at a sacrifice of cost per unit; it may be required to secure a maximum rate of income relative to the capital invested; or it may be required to secure the maximum net annual income from the stream in question. The problem of economy presents itself in a great variety of forms, and each requires different treatment. Of the cases just cited, each will be seen to be different from every other, and it is only to the last mentioned that the author's solution applies.

Mr. Adams offers as a major premise the self-evident statement that:

"The relation between the investment in any pipe line and the value of the energy lost in frictional resistance to flow is most economical wherein the cost of the pipe plus the value of the energy sacrificed is a minimum."

The condition for a minimum value of cost of pipe plus the value of lost energy is then determined, and the conclusion is drawn as follows:

"That pipe fulfills the requirements of greatest economy wherein the value of the energy annually lost in frictional resistance equals four-tenths (0.4) of the annual cost of the pipe line."

The inference is that economy in the relation of the investment in the pipe to the value of energy lost in frictional resistance is identical with economy in power development. The author does not seek to establish this identity in his paper, although it is essential to the logical development of his conclusion from the premises. If the conditions are as in the fourth case cited by the writer, namely, a desire on the part of the client to secure a maximum net annual earning capacity from his water rights, then, and then only, can this identity be established. The net annual value of the power delivered by the pipe line will be equal to the annual value of the total power of the stream—after deducting the annual cost of the impounding and head works—less the annual cost of the pipe plus the annual value of the power sacrificed. Therefore, in so much as the pipe affects the result, the annual net earnings will be a maximum when the latter mentioned sum is a minimum.

Mr. Stevens.

That this solution does not apply to the first case cited by the writer, may be seen by an inspection of Table 1. It will be noted that the cost of pipe line per horse-power developed is least for the smallest pipe and greatest for the largest, being nearly five times as great for the latter as for the former. In such a case, the pipe cannot be considered alone in determining conditions for greatest economy in cost for a definite horse-power, but must be taken in connection with the other works required, and its diameter must be adjusted to the head to be used.

In order to protect the busy engineer from the possibility of making a mistaken application of any rule of economy, the rule should be stated so as to define the nature of the economy effected. The valuable rule derived by Mr. Adams, following this idea, would result in a statement about as follows: In developing a water-power of fixed fall and flow, that pipe has the greatest net annual earning capacity wherein the value of the energy lost in frictional resistance is equal to four-tenths (0.4) of the annual cost of the pipe line.

It is the writer's opinion that the author's solution of this problem should give him no cause to regret the "loss" of his calculus. The laws of graphics are more easily retained in one's memory than those of the calculus. An engineer should be economical of his own mental energy as well as of his client's resources.

Mr. DeMott.

C. L. DeMott, Assoc. M. Am. Soc. C. E. (by letter).—In the main, Mr. Adams has given some well-considered points, but the writer believes that in some instances he is misleading and incorrect.

The statement of the solution of the problem seems to be logical and correct, but the two statements—one as to the total cost and the other as to yearly cost—appear to be confused in his graphical solution.

By referring to the diagram, Fig. 1, it appears that the most economical pipe would have a diameter less than 48 in., with steel at 7 cents per pound and "energy at $100 (per year) per horse power." By referring to Column 7 of Table 1, it appears that a 66-in. pipe would cost but $6 170 per thousand feet more than a 60-in. pipe. Now, if the allowance of 10% (used by Lord Kelvin) be used for interest and depreciation, this will give an increase in yearly cost of $617; while, from Column 8 of Table 1, it is noted that the annual saving in energy is $680. Whence it appears that a 66-in. pipe is more economical than a smaller one.

This conclusion is borne out by a mathematical solution of the problem. Using the statements and nomenclature of Mr. Adams:

Yearly cost of pipe $= a D^2$,

Yearly value of lost energy $= m S$,

Mr. DeMott.

where a and m are constants, and D is the diameter of the pipe, in feet. Substituting in the value of S the value of V in terms of D and the given quantity of water, we have:

$$\text{Yearly value of lost energy} = \frac{b}{D^5},$$

where b is a new constant.

Hence, by the conditions, $\frac{b}{D^5} + a\,D^2$

must be a minimum, or, $-\frac{5\,b}{D^6} + 2\,a\,D = 0.$

Whence, $$D = \sqrt[7]{\frac{5\,b}{2\,a}}.$$

From the values used by Mr. Adams,

$$a = 115,$$

$$\text{and } b = 7\ 500\ 000,$$

which gives $$D = 5.564 \text{ ft.}$$

This problem, however, can be solved most readily by the "cut and try" method. The pipe manufacturers use only certain thicknesses of metal, and make certain shop sizes of pipe. After calculating the cost of two sizes of pipe, which are thought to be most nearly correct, and making a comparison of the value of the energy saved, a pretty fair indication of the most economical size will be given. An hour's additional work will solve the problem.

If Mr. Adams will plat his curves representing the yearly cost of pipe, and the yearly value of wasted energy, on the same axis, he will observe that the least value, of the curve formed by adding the two sets of values, will be quite near the point where the separate values are equal, the method perhaps used by Lord Kelvin in the solution of his problem.

Mr. Kuichling.

E. KUICHLING, M. AM. SOC. C. E.—This subject is of much interest, and the author has presented a mode of solution that seems to be very simple. His fundamental proposition is that the most economical diameter of a pressure pipe is the one which will make the sum of the total cost of the pipe line and the value of the head lost in overcoming the frictional resistances in such pipe a minimum; and, without any further explanations and definition of what is meant by the value of the head lost by pipe friction, he states that this proposition is self-evident.

In the speaker's opinion, the proposition is not self-evident, but requires considerable thought; and as to the value of the power lost by pipe friction, much room for explanation is open. It may be inferred that the value of $100 and $50 per horse-power, used by the author in Table 1 and his diagram, Fig. 1, is the capitalized value of the difference between the price paid by the consumer per

horse-power per year and the cost of the operating expenses, maintenance and depreciation for producing one horse-power per year with the plant under consideration. This difference represents the annual interest and profit on the investment of capital or cost of construction per horse-power. If the aggregate of such interest and profit be taken at 10%, and the difference between the selling price and said cost of production per horse-power per year is \$10, the value of each horse-power lost by pipe friction will be \$100; and, similarly, if that difference is \$5, the value of each horse-power so lost will be \$50. Mr. Kuichling.

With this definition of the value of each horse-power capable of development at the given site the speaker is in accord, but as the difference between said price and cost is by no means constant in different localities, a uniform figure cannot be adopted. The value of the power lost by pipe friction, therefore, will vary with the special conditions pertaining to each plant, and must be estimated after a plan for the work has been made and its approximate cost and prospective revenue have been ascertained.

When the available quantity of water, Q, in cubic feet per second, is fixed, the determination of the most economical diameter, d, for the pipe becomes essentially a problem of transportation, into which only two elements enter. The first is the cost of the pipe line, which is obviously a certain function of the diameter, and the second is the capitalized cost of overcoming the resistances to the transportation or motion of Q, which is the value of the power lost by the friction head, h_x. The virtual cost, U, of transporting Q through the pipe is the sum of these two costs; and as the friction head, h_x, can be expressed in terms of Q and d, it follows that U is a definite function of d. The problem is then to find that value of d which will make U a minimum.

The cost of the pipe conduit per linear foot, as given in Table 1, is very closely expressed by the formula:

$$y = 0.9043d^2 + 2.3076d - 4.592, \text{ in dollars,}$$

wherein d is the diameter of the pipe, in feet; and, if the length is denoted by l, the cost of the conduit will be $l\,y$ dollars. Based on the Chezy formula, the friction head is $h_x = \left(\frac{8}{\pi}\frac{Q}{c}\right)^2 \frac{l}{d^5}$, and with a turbine of 80% efficiency this friction head represents the expenditure of $N_x = \frac{Q\,h_x}{11}$ horse-power for moving Q through the pipe; and if the capitalized value of the energy so expended is at the rate of S dollars per horse-power, as aforesaid, its virtual cost will be $= S\,N_x$ dollars. The entire cost of transporting Q, therefore, is:

$$U = l\,y + S\,N_x = l\,(\alpha\,d^2 + \beta\,d - \gamma) + \frac{S\,Q}{11}\left(\frac{8}{\pi}\frac{Q}{c}\right)^2 \frac{l}{d^5} \cdots (1)$$

Mr. Kuichling.

To find the value of d which will make U a minimum, place the first differential coefficient of U with respect to d as a variable equal to zero, thus obtaining, finally:

$$d = \left[\frac{2.9475\ S}{(2\ \alpha\ d + \beta)\ c^2}\right]^{\frac{1}{6}} \sqrt{Q} \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

It will be noticed that d also appears in the denominator of the second member of Equation 2, and hence the solution must be obtained by trial. As a preliminary value for d in the second member, assume that which corresponds to a velocity of about 6 ft. per sec., or $d = 0.46\ \sqrt{Q}$, and thence deduce a first approximation by solving the equation. The result may then be used in the second member, thus obtaining a second approximation which may be sufficiently accurate for all practical purposes.

For the case given by the author, we have $\alpha = 0.9043$, $\beta = 2.3076$, $c = 100$, $Q = 100$, and $S = 100$ as one value and 50 as another. Taking $d = 0.46\ \sqrt{Q} = 4.6$ as a preliminary value for use in the second member of Equation 2, there follows with $S = 100$: $d = 0.375\ \sqrt{Q}$ as a first approximation, and thence $d = 0.384\sqrt{Q}$ as a second approximation. With $S = 50$, and a preliminary value of $d = 3.5$, there follows $d = 0.346\ \sqrt{Q}$ as a sufficiently close approximation.

If the cost of the pipe conduit per linear foot be expressed by $y = \alpha\ d^2$, as contemplated by the author, the value of α will be approximately 1.1667, according to Table 1. Equation 1 will then become:

$$U = l\ \alpha\ d^2 + \frac{S\ Q}{11}\left(\frac{8\ Q}{\pi\ c}\right)^2 \frac{l}{d^5} \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

and by placing the first differential coefficient equal to zero, as before, there follows:

$$2\ l\ \alpha\ d - \frac{5\ S\ Q}{11}\left(\frac{8\ Q}{\pi\ c}\right)^2 \frac{l}{d^6} = 0,$$

or

$$2\ [l\ \alpha\ d^2] = 5\left[\frac{S\ Q}{11}\left(\frac{8\ Q}{\pi\ c}\right)^2 \frac{l}{d^5}\right]$$

or $2[\text{cost of pipe conduit}] = 5[\text{value of head or energy lost}]$, whence the author's proposition that the most economical diameter is that for which the value of the head or energy lost by frictional resistances is four-tenths of the cost of the pipe conduit.

From the last equation we also find the required value of the diameter in terms of S and Q:

$$d = \left(\frac{1.4738\ S\ Q^3}{\alpha\ c^2}\right)^{\frac{1}{7}} = \left(\frac{1.4738\ S}{\alpha\ d\ c^2}\right)^{\frac{1}{6}} \sqrt{Q} \quad \ldots\ldots\ldots (4)$$

The foregoing demonstration makes the economical diameter, d, in any given case, depend mainly on the value of Q, which is

more convenient for practical use. For $\alpha = 1.1667$, $c = 100$, and $S = 100$ and 50, we will have, respectively: $d = 0.536Q^{\frac{3}{7}}$ and $0.485\ Q^{\frac{3}{7}}$; and with $Q = 100$ there follows, respectively: $d = 3.8541$ and 3.4908, or substantially the same as found above with a different expression for the cost of the pipe conduit. Mr. Kuichling.

It may also be added that the cost, y, of the pipe conduit per linear foot, as given above, is not generally applicable, but should be regarded as adapted only to a particular locality. In other cases, where the fall is different, the values of the factors, α and β, will be greatly modified.

WILLIAM J. ROBERTS, ASSOC. M. AM. SOC. C. E. (by letter).— The author has presented a "rule defining the true economic relation to be observed in designing pipes for high-pressure water-power installations * * * the volume of discharge * * * being fixed," that may be verified readily when the equation of the curve is known. Mr. Roberts.

This equation may be deduced as follows:

Cost of pipe per foot, steel at \$0.07 per lb. $= 1.17D^2 - \frac{7.7}{D^3}$, in which D is the diameter, in feet, the result being expressed in dollars.

For velocities below 10 ft. per sec., the final term may be neglected without sensible error. The equation then becomes:

$$x = 1.17D^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

The energy lost in 1 000 ft. of pipe $= 0.4\ \frac{v^2}{D} = \frac{6\ 500}{D^5}$.

The value of the energy lost in 1 000 ft. of pipe at \$100 per horse-power $= \frac{6\ 500}{D^5} \times \frac{100}{8.8} \times 100 = \frac{7\ 380\ 000}{D^5}$.

For convenience, this may be written:

$$y = \frac{7\ 380\ 000}{D^5} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

Eliminating D between Equations 1 and 2, there results:

$$x^{\frac{5}{2}}\ y = 10\ 900\ 000 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

which is the required equation of the curve.

Differentiating this equation:

$$\frac{5}{2}\ x^{\frac{3}{2}}\ y\ d\ x + x^{\frac{5}{2}}\ d\ y = 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Whence

$$\frac{d\ y}{d\ x} = -\frac{5\ y}{2\ x} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

which is the slope of the tangent to the curve at any point the coordinates of which are x and y. This tangent makes an angle of 135° with the x-axis.

Mr. Roberts.

The tangent of 135° is —1, hence,

$$\frac{d\,y}{d\,x} = -\frac{5\,y}{2\,x} = -1 \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

$$5\,y = 2\,x \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

$$y = 0.4\,x \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

which was to be proven.

The coefficients and constants in Equations 1, 2, and 3 are based upon pipe at 7 cents per pound, and energy at $100 per horse-power, their values being determined with the slide-rule. Inasmuch as the character of the curve in Equation 3 is independent of the magnitude of the constant, this form of the equation will apply to any price for pipe, or any value of the energy lost. This is the more readily seen in Equation 4, where the second member becomes zero.

Hence the rule is established.

Messrs. Marx and Wing.

CHARLES D. MARX and C. B. WING, MEMBERS, AM. SOC. C. E. (by letter).—The following is a mathematical verification of the rule formulated by Mr. Adams:

It is assumed that the yearly income will vary as the horse-power output per annum multiplied by the value per horse-power at the generating station, less the interest on the cost of the pipe line.

Let I = the annual income;
h = the total head;
h_1 = the head lost in friction;
d = the diameter of the pipe;
r = the rate of interest per annum;
k = receipt per horse-power per year.

Then, the total horse-power delivered at the generating station $= f(h—h_1)$, in which f is a constant. The weight, and therefore the cost, of the pipe will vary approximately as $f_1 d + f_2 d^2$ (in which f_1 and f_2 are constants) as we cannot start with zero as the thickness of the pipe.

The head lost in friction will vary as $\frac{f_3}{d^5}$, in which f_3 is a constant.

Therefore,

$$I = k f (h - h_1) - r (f_1 d + f_2 d^2).$$

$$I = k f h - \frac{f f_3 k}{d^5} - r f_1 d + r f_2 d^2.$$

$$\frac{d\,I}{d\,d} = \frac{5 f f_3 k}{d^6} - r f_1 - 2 r f_2 d.$$

Equating to zero:

$$\frac{k f f_3}{d^5} = \frac{1}{5} r f_1 d + \frac{2}{5} r f_2 d^2,$$

or, approximately, the capitalized value of the annual loss of horse-power should be four-tenths of the cost of the pipe line. Messrs. Marx and Wing.

JAMES H. WISE, ASSOC. M. AM. SOC. C. E. (by letter).—The subject of this paper is of considerable interest and great importance in the design of high-pressure pipe lines for hydro-electric installations. This problem was given careful consideration by F. G. Baum, Assoc. M. Am. Soc. C. E., formerly Chief Engineer of the California Gas and Electric Corporation, and the writer, in considering the design of pressure pipe lines installed by the above corporation. A mathematical analysis of the problem (herewith submitted) was made, the result agreeing with that obtained graphically by Mr. Adams. Mr. Wise.

When this problem was originally taken up, the general formula, $h = \frac{f\,l\,v^2}{2\,g\,d}$, for head loss was used; but, for ease of comparison in this discussion, the equation, $S = \frac{V^2}{C^2\,R}$, will be used; the deduction, of course, being the same. For obtaining a simple expression for power loss, due to pipe friction, the following equations and assumptions were taken:

$S = \frac{V^2}{C^2\,R}$, Friction head loss in the pipe;

C = Pipe coefficient, taken as constant;

$R = \frac{D}{4}$, Hydraulic radius;

$V = \frac{Q}{A}$, Velocity, in feet per second;

Q = Quantity of water, in cubic feet per second;

D = Inside diameter of the pipe;

$A = \frac{\pi\,D^2}{4}$, Area of the pipe.

Eliminating all variable factors except D, by substitution in terms of D, we have, $S = \frac{64\,Q^2}{C^2\,\pi\,D}$; but the power loss, $P = Q\,\rho\,S$, where $\rho = 62.5$ lb., the weight of 1 cu. ft. of water. If e is the value of power per foot-pound per second, for one year, the value of the annual power loss, L, may be expressed as $L = e\,P = \frac{64\,\rho\,e\,Q^3}{C^2\,\pi\,D^5}$; but, as all terms except D in the last member of the equation may be considered constant, the equation may take the form, $L = \frac{K}{D^5}$.

To obtain the annual cost of the pipe, I, the following assumptions were made:

Mr. Wise.

t = Thickness of metal;
πD = Circumference of pipe;
w = Weight of metal per cubic inch;
i = Annual interest cost of metal per pound.

Therefore, $I = \pi D t w i$, but, for any pressure, the thickness of the pipe varies as the diameter, or, in the form of an equation, $t = m D$, where m is a constant depending upon head and safe stress assumed. Eliminating t, we have $I = \pi D^2 m w i$, but all terms in the second member of the equation are constant except D; the equation, therefore, may be expressed, $I = N D$, N being a constant.

The problem now is to make T, the annual cost of the pipe, I, plus the annual value of the power loss, L, a minimum, or, expressed as an equation, $T = I + L = N D^2 + \frac{K}{D^5}$, to be a minimum; but, by the familiar rule of calculus, this obtains when the first derivative of T with respect to D is equal to zero.

$$T = N D^2 + \frac{K}{D^5}.$$

$$\frac{d T}{d D} = 2 N D - \frac{5 K}{D^6} = 0,$$

or

$$2 N D = \frac{5 K}{D^6}.$$

Multiplying both sides of the equation by $\frac{D}{5}$, we have $\frac{2}{5} N D^2 = \frac{K}{D^5}$. Therefore, T is a minimum when $\frac{2}{5}$ or 0.4 of the annual cost of the pipe equals the yearly value of the power loss; or, it may be expressed in words, as Mr. Adams has done in his paper.

It is found, invariably, that in the mathematical discussion of practical problems, assumptions must necessarily be made which will sacrifice theoretical exactness, and, in connection with this, the writer calls attention to the fact that, in addition to assuming C to be constant for all diameters of pipe, in using D for the inside as well as the mean diameter of the pipe and other approximations, it is also making a broad assumption to state that the cost of riveted steel pipe is proportional to the square of the diameter; nor does the over-weight due to lap straps and rivet heads bear a constant relation to the weight of the net pipe cylinders. In this it may be noted that with thin metal, say $\frac{7}{16}$ in. or less, where a lap joint of about 70% efficiency is used, the over-weight due to laps and rivet heads may vary from 4% to 15%, depending upon the diameter of the pipe, while, with thick metal, say about $\frac{7}{8}$ in., where a triple-riveted, double-welt, butt joint, developing about 85% of the plate, is generally used, the over-weight due to straps and rivet heads may

vary from 20% to 45%, depending upon the diameter of the pipe. It may be noted, further, that the price per pound is not constant, it being less for large pipes of thin metal than for small pipes of thick metal. This is largely due to the higher shop costs in rolling and assembling heavy sheets. Mr. Wise.

E. W. MYERS, ASSOC. M. AM. SOC. C. E. (by letter).—In order that the rule, as given, or any rule, for the economic size or cost of a pressure pipe may be true, it must also be true that the plant under construction shall be able to develop and use all the energy which the given stream or quantity of water is capable of developing. In other words, it must be a commercial necessity to obtain from the given quantity of water and the available head the greatest economical amount of power; and the problem which the author set for himself is the determination of the size of pipe which will furnish this power with the maximum degree of commercial economy. This size is that of which the annual cost plus the annual value of the power lost in the pipe is a minimum. Mr. Myers.

This is a modification of a law regarding the electrical transmission of energy first proposed by Sir William Thomson in 1881, and, as originally formulated, stated in effect that the most economical area of conductor will be that for which the annual interest charge equals the annual cost of the energy lost in it. More modern investigators of the problem have modified this law to read as given by the author.

In the case of an electrical conductor, the application of this rule usually produces a line of greater first cost than is generally justified by the conditions, for the presence of a little extra power brings no profit unless it can be sold, and the same thing is true of a water-power plant. The pipe is the conductor in this case, and unless all the power transmitted by the pipe can be used, then the size of pipe which will be given by the rule proposed is not the economical size, and the cost not the true economic cost, and, in a great many instances, water-power plants are not sold up to their full capacities.

The rule, therefore, applies only when the water-power is to be developed to its absolute maximum economic capacity, and when this capacity is to be used. Each plant must be considered on its merits as a separate and distinct proposition, and various conditions, other than that of absolute maximum economy, are apt to determine the allowable loss of head in each case.

In any water-power development there are a number of viewpoints from which the matter of economy may be regarded. Some of the most apparent are:

(1).—It may be necessary to make the development to yield the maximum amount of power, somewhat irrespective of the first cost

Mr. Myers. of the plant, as when the water supply is fixed or limited, and the power is imperatively required. Maximum efficiency is the desideratum in this case.

(2).—The water supply and head may be ample and the requirements for power limited. Such a condition frequently occurs in connection with manufacturing establishments, and the problem then consists in developing enough power for the comparatively fixed needs of the plant. In such a case the lowest practicable first cost is to be sought, giving the minimum first cost per horse-power.

(3).—It may be required to make such a development that the annual cost of operation will be a minimum for a maximum output, so that the annual net income from the plant shall be a maximum, which is the phase of the problem considered by the author, and which, with the limitation imposed—that all the power developed can be utilized—has been solved by him as far as the consideration of the pipe alone as a factor in the annual cost of operation is concerned. The pipe line, however, is only one of the items which go into the make-up of a water-power plant, and, frequently, it is only a small proportion of the total cost, therefore it must be considered in connection with other items of the development cost in determining the economy of operation of the plant.

Mr. Toll. ASAHEL C. TOLL, JUN. AM. SOC. C. E. (by letter).—The author expresses the hope that someone will verify, by analytic methods, his rule:

"That pipe fulfills the requirements of greatest economy wherein the value of the energy annually lost in frictional resistance equals four-tenths (0.4) of the annual cost of the pipe line."

In this discussion the writer has tried merely to solve the problem with the author's assumptions as a basis, except that it is assumed that the weight of the pipe is proportional to the square of the diameter (not taking account of the changes in pressure due to frictional losses), and that the cost is proportional to the weight. That is, the cost equals $m\ d^2$, in which d is the diameter and m is a constant.

If the frictional loss equal $\frac{l\ v^2}{c^2\ r}$, as the author assumes, (l being the length of the pipe, v the mean velocity, c the friction coefficient, and r the hydraulic radius), then, since $r = \frac{d}{4}$ and $v = \frac{4\ q}{\pi\ d^2}$, ($q$ being the discharge), the frictional loss equals $\frac{64\ l\ q^2}{\pi^2\ c^2\ d^5}$. The value of this loss equals $\frac{64\ l\ q^2}{\pi^2\ c^2\ d^5} \times \frac{w\ q\ h}{550}$, in which w is the weight of the unit volume of water and h is the value of the energy per horse-power. In the foregoing expression all the factors except d are constant, there-

fore it may here be represented by the expression, $\frac{n}{d^5}$, n being a constant. Mr. Toll.

Thus we have the value of energy lost, which we will call a, and the cost of pipe (say b), in terms of d; and

$$a + b = \frac{n}{d^5} + m\,d^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

When this expression is a minimum, its first derivative equals zero. Differentiating and equating to zero, we have

$$2\,m\,d - \frac{5\,n}{d^6} = \text{zero, or } d = \sqrt[7]{\frac{5\,n}{2\,m}} \quad \ldots\ldots\ldots\ldots (2)$$

Substituting in Equation (1), we have

$$a + b = \frac{n}{\left(\sqrt[7]{\frac{5\,n}{2\,m}}\right)^5} + m\left(\sqrt[7]{\frac{5\,n}{2\,m}}\right)^2 \quad \ldots\ldots\ldots\ldots (3)$$

Therefore, the ratio of the value of the lost energy to the cost of the pipe, for greatest economy, becomes

$$\frac{a}{b} = \frac{n}{m\left(\sqrt[7]{\frac{5\,n}{2\,m}}\right)^{5+2}} = \frac{n}{m\left(\frac{5\,n}{2\,m}\right)} = \frac{2}{5} \quad \ldots\ldots\ldots\ldots (4)$$

If the weight of the pipe varies with the pressure head, as the author suggests, the foregoing equations become quite complicated, and to the writer it did not seem to be worth while to work out their solution. It may readily be seen, however, that as pressure head increases with increase of diameter (the discharge remaining constant), the effect of this factor would be to make the economic diameter slightly smaller, and, therefore, to increase slightly the above ratio of the value of the lost energy to the cost of the pipe.

F. G. Baum, Assoc. M. Am. Soc. C. E. (by letter).—In the discussion of this subject, the value of one horse-power per year of the pipe losses has been generally taken at $100, although Mr. Adams calculates one case for $50. If a value of $100 is given to the pipe losses, it really gives a value of $\frac{100}{0.7} = \$143$ per horse-power on the line (or $72.50 for the $50 basis) on account of inefficiency of conversion. The first figure gives about 2.2 cents per kw-hr., and the second 1.1 cents per kw-hr. Both these figures are too high, and the writer calls attention to this fact, particularly because a great many engineers make reports basing their conclusions on an entirely erroneous idea of the value of power. Of course, pipe lines or any other structures calculated with an erroneous value of power will give wrong results. A check must be made in order to be certain that the pressure drop is not too large for the best results on the water-wheels. Mr. Baum.

Mr. Adams. ARTHUR L. ADAMS, M. AM. SOC. C. E. (by letter).—The writer is grateful to the authors of the several excellent mathematical verifications of the rule set forth in the paper, as well as to several others whose demonstrations have come to him direct.

While the methods of attack are naturally different, all are in accord in result save Mr. DeMott, who seems to have fallen into error in using the figures in Column 8 of Table 1 as annual value when they represent capital or full value. Mr. Baum also makes the same mistake of construing as annual value what the writer intended as full value.

In the preparation of the paper, the writer was not oblivious to the difficulties to be encountered in determining the value of the energy consumed in frictional resistance within the pipe. This, however, is another question, and does not affect the merit of the rule enunciated.

If there is room for doubt as to the writer's thought in using the term "economical," Mr. Stevens and Mr. Myers have removed it.

The wide variation in weight of seams and overlaps suggested by Mr. Wise obtains only when comparing extremes in diameters. This variation virtually disappears within the range considered in designing a pressure pipe in accordance with the rule. The same reply may be made to Mr. Wise's suggestion as to variation in price per pound.

Both Mr. Buck and Mr. Butcher, in their mathematical analyses, have deduced formulas which simplify the application of the rule, while the excellent diagram presented by Mr. Buck renders the application very easy indeed.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 1055.

THE REINFORCED CONCRETE BRIDGE ACROSS THE HUDSON RIVER AT SANDY HILL, NEW YORK.*

By William H. Burr, M. Am. Soc. C. E.

With Discussion by Messrs. Henry H. Quimby, C. L. Slocum, D. W. Krellwitz, E. W. Stern, and William H. Burr.

The bridge across the Hudson River at Sandy Hill, N. Y., is a reinforced concrete structure built jointly by the Counties of Saratoga and Washington and the Union Bag and Paper Company, whose extensive paper and pulp mills are located on both sides of the river in close proximity to the bridge. The structure is to serve for both highway and passenger traffic and for the transportation of materials, either raw or finished, between the mills and the railway tracks on either side of the river.

The width of the bridge, between the outer surfaces of the spandrel walls, is 35 ft. 8 in., while the width between the coping stones is 32 ft. The latter width affords room for a roadway 26 ft. 9 in. wide and a 5-ft. 3-in. sidewalk. On one side of the roadway is placed a standard gauge railway track, the center of the nearest rail being 4 ft. 3 in. from the adjacent coping. The railway track is designed for the use of the Union Bag and Paper Company in the transportation of its material, to which reference has already been made.

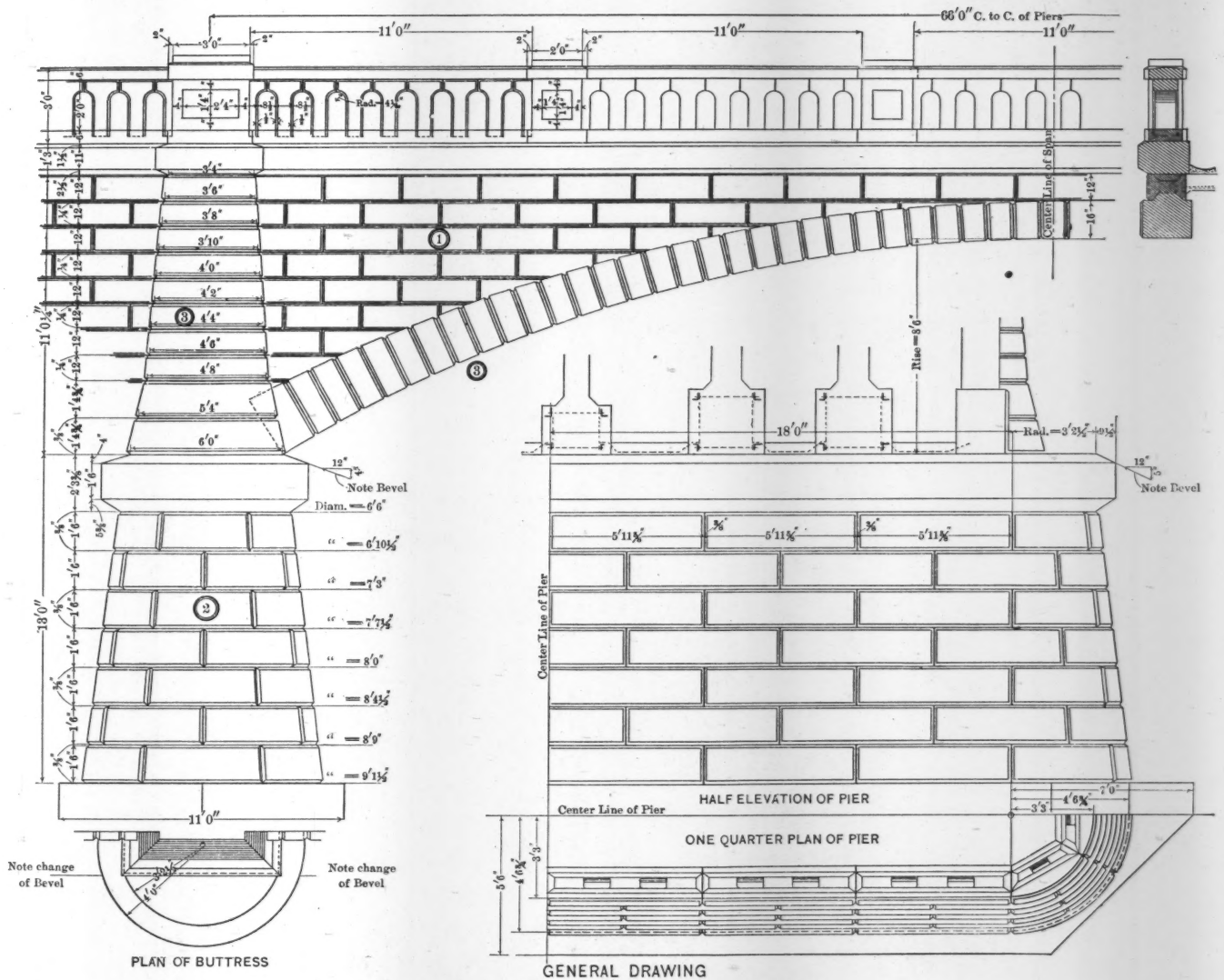
* Presented at the meeting of May 15th, 1907.

There are fifteen arch spans, each 60 ft. in the clear, the piers being 6 ft. thick at the top between the springing joints of the arch ring, and 9 ft. thick 13 ft. below the springing joints at the top of the foundation mass of concrete. The total length of the bridge and the two abutment walls is 1 025 ft., making, it is believed, the longest reinforced concrete structure in existence at the present time. The radius of the intrados is 57 ft. 6 in. As shown on the plans, the rise of each arch is 8 ft. 6 in. The depth of the ring stones is 18 in. at the crown and 30 in. at the skewback or springing joint.

The spandrel walls, as well as the arch ring and the exterior walls of the piers and abutments, are composed of concrete blocks designed with interior open spaces or voids filled with thin mortar or thick grout after being laid in place. These blocks were cast or moulded in wooden forms in a building near the east end of the bridge, and were usually taken out of the forms in from 24 to 48 hours after being cast. Reference to this feature of the structure will be made later.

The railing is also of concrete, the upright members having been cast in forms or moulds in a manner similar to that in which the spandrel and other blocks were made, with the horizontal members cast in place.

The main supporting members of each span are five arch ribs, the two heavier ribs being placed under the railway track, each rail being over the center of a rib. The other three ribs are placed equi-distant from each other and from the adjacent railway rib, 5 ft. 10 in. from center to center. Internal spandrel walls, each 12 in. thick for the three lighter ribs, and 15 in. thick under the railway track, support a reinforced concrete floor plate or slab 8 in. thick for the entire roadway and 6 in. thick for the sidewalk, the top surface of the latter being 8 in. above the former. The top surface of the concrete is the finished sidewalk surface, but an asphalt pavement, crowned to a thickness of 4 in. between the railway tracks and 2 in. at the gutter, is laid for the roadway pavement, bricks suitably formed being laid on each side of each rail. The outer edges of the sidewalk plates and their proportionate loads are carried by the spandrel walls and the ring stones. The structure is designed to carry its own weight, a snow



GENERAL DRAWING
OF BRIDGE OVER THE HUDSON RIVER AT SANDY HILL, N. Y.
FOR THE COUNTIES OF SARATOGA AND WASHINGTON.

load of 12 lb. per sq. ft. and a moving load of 100 lb. per sq. ft. over the roadway and sidewalk, while the ribs under the railway track are designed for a moving load consisting of a train of cars, each, with its load, weighing 120 000 lb., carried on two trucks with four wheels each. The reinforced concrete roadway floor plates are also designed to carry a truck weighing 12 tons resting on a wheel base of 10 by 6 ft.

In the computations the modulus of elasticity of concrete was taken at 2 000 000 lb. and that of steel at 30 000 000 lb. per sq. in.

The bridge is designed to carry the preceding loads without exceeding a compressive stress of 500 lb. per sq. in. in the concrete. For all ordinary loadings, even with the railway loads prescribed above, that compressive stress does not exceed about 400 lb. per sq. in. The tensile resistance of the concrete in any part is not taken into account. As shown on the plans, the cross-section of the ribs under the railway tracks is 3 ft. square at the springing joints and 3 ft. by 1 ft. 10 in. at the crown, while the three lighter ribs are 2 ft. 8 in. by 2 ft. 3 in. at the springing joint and 2 ft. 8 in. by 1 ft. 2 in. at the crown. The steel reinforcement of each rib consists of four 3 by $2\frac{1}{2}$ by $\frac{5}{16}$-in. angles embedded in the concrete, as shown on the plans, with not less than 2 in. of concrete outside of them at all points. These reinforced angles are latticed into their proper relative positions by $2\frac{1}{4}$ by $\frac{5}{16}$-in. steel lattice bars. It was found that these reinforcing ribs could not be secured in the shape of ordinary riveted members without great delay and excessive cost, and therefore the steel angles and lattice bars were shipped directly from the mills to the bridge site, where a portable punch, shears, and simple bending rolls were improvised. With this simple field outfit, manned by three or four laborers, all the reinforcing steel required for the ribs was laid out, punched, bolted together and put in place. As all this work of fabrication was done at the bridge site, all the work was bolted up, no rivets whatever being used. The plans show how rigidly the steel reinforcement of the ribs was joined at the tops of the piers with a corresponding reinforcement running down to the bottom of the concrete hearting of the piers. By these means the ribs and piers were made firmly continuous with each other. Furthermore, the batter of the sides of the piers not only

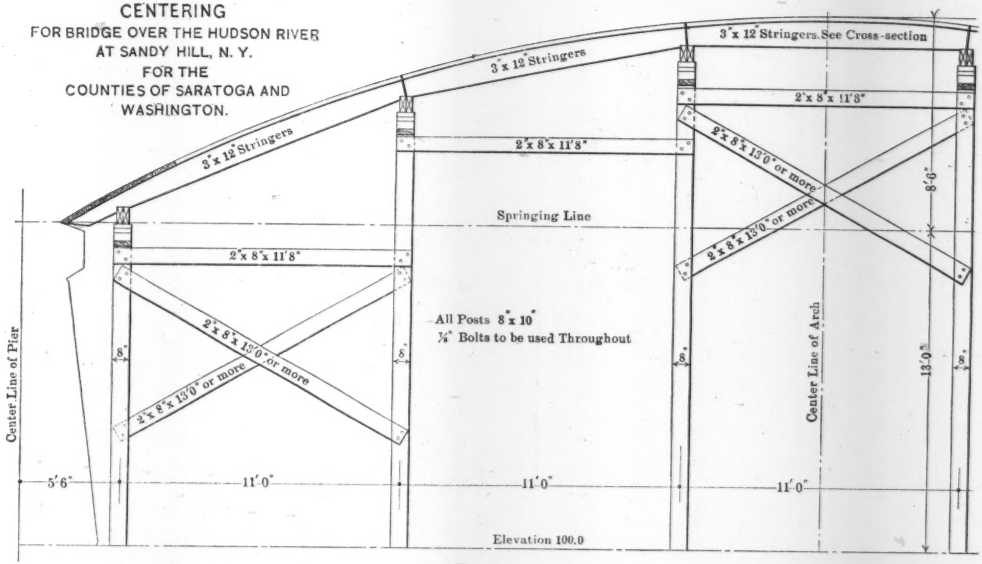
results in abundant stability when a single span only is fully loaded, with no live load on the adjacent spans, but it also results in a pleasing visual effect.

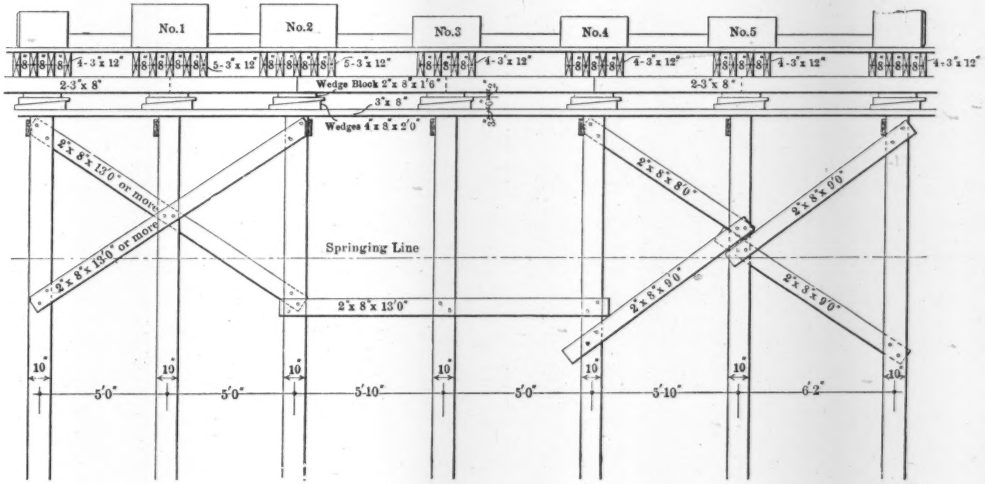
In order to lighten the structure as much as possible and to economize in material, panels of sheeting 5 ft. long and 6 ft. apart are omitted between the highway ribs, but the sheeting is made complete between the two ribs under the rails of the railway. The sheeting itself, wherever used, is 6 in. thick; a thickness of 4 in. would probably have been sufficient, but, as the bridge is partly for a rather heavy railway traffic, and as the open spandrel spaces beneath the floor may be used eventually for pipes and conduits, it was considered advisable to adopt a thickness of 6 in.

The steel reinforcement for the entire work, exclusive of the ribs, consists of Thacher rods. The floor slabs or plates are reinforced both transversely and longitudinally of the structure by $\frac{3}{4}$-in. rods, 6 in. apart longitudinally, while those parallel to the axis of the bridge are placed as fully shown on the plans. The reinforcing rods for the roadway and sidewalk lap past each other, so as to make a continuous plate from one side of the bridge to the other. Further than this, the ends of the sidewalk rods extend about half way through the coping, being bent so as to increase the firmness of hold. This entire arrangement makes a continuous reinforced structure, at the roadway elevation, of spandrel walls, roadway and sidewalk plates. The interior spandrel walls have embedded in them, throughout the mass of their higher portions, short $\frac{3}{8}$-in. rods spaced 12 in. apart on centers. Near the piers these walls are fully 9 ft. high, and only 12 in. thick under the roadway. Although their thickness under the railway is 15 in., it was considered advisable to supply positive resistance against any tendency to cleavage or splitting in any direction approximately vertical. The plans also show the thorough method of anchorage used for producing a positive bond between the soffit plates or sheeting and the ring stones. This particular steel reinforcement, with the transverse rods running through the sheeting, insures an absolutely firm and continuous bond between the ring stones under the exterior spandrel walls and the sheeting, together with the interior spandrel walls. An examination of the plans will show that while the structure is essentially of skeleton

CENTERING

FOR BRIDGE OVER THE HUDSON RIVER
AT SANDY HILL, N. Y.
FOR THE
COUNTIES OF SARATOGA AND
WASHINGTON.





character, with economical interior vacant spaces, its parts are rigidly bound together in the most positive manner with the steel reinforcement and by continuity of concrete work, the steel being nowhere within 2 in. of the exterior surface of the concrete.

The concrete blocks of the piers and abutments are also rigidly bonded or tied to the interior concrete hearting with the flat steel Z-anchors reaching at least 18 in. into the concrete backing. The piers and ribs, with the steel reinforcement of each, are designed and built so as to constitute a reinforced monolithic concrete mass, insuring a fixedness of the ends of the ribs to the pier with such a bond as to make impossible any sensible movement of one with reference to the other.

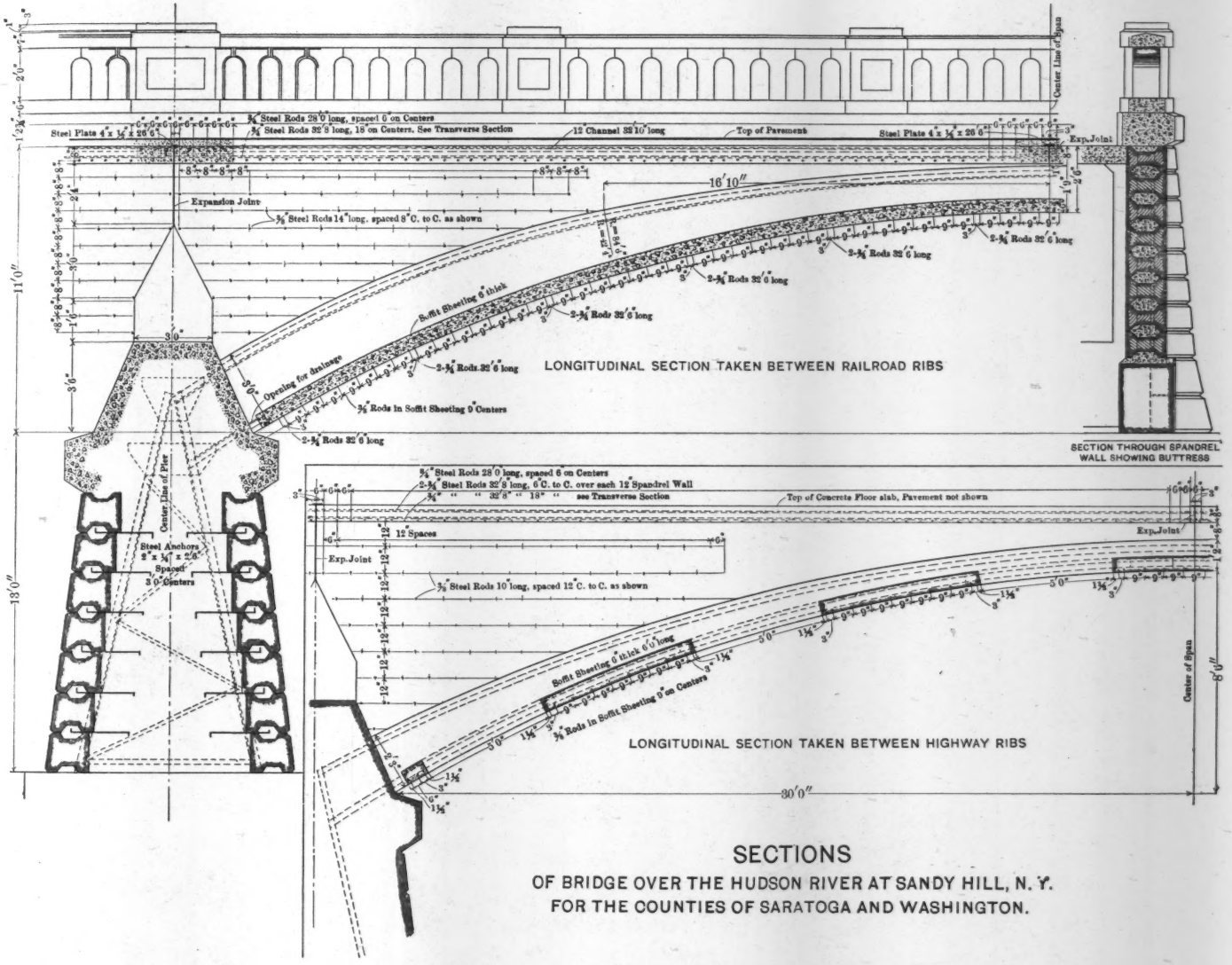
The system of designing and casting the blocks forming the exterior spandrel walls, arch rings and the exterior walls of the piers and abutments constitutes a most interesting feature of this work, if not its only really novel one. As the plans show, these blocks are formed with interior vacant spaces and with corresponding channels or recesses along their beds and joints. When these blocks, therefore, are laid in place, as in the process of construction, a honeycomb mass with internal channels or open spaces is formed. The outer portions of the horizontal and vertical joints may then be packed with any convenient or economical material such as jute, or any other fibrous material, or paper waste. After thus packing the outer portions of the joints, mortar or thick grout may be poured into the openings so as to fill completely all portions of these interior spaces. It will be observed that the recesses in the blocks and other internal openings were formed so as to insure a thorough and complete filling of every portion of the joints and spaces in the blocks as cast. These recesses or internal spaces can be formed quickly and cheaply by the suitable use of partially filled paper bags of sand, as was done in this case. After the block is set these paper bags are punctured and the sand at once runs out, leaving the desired voids. Furthermore, this process makes every wall thus formed essentially monolithic, resulting in the highest obtainable degree of strength, solidity and water-tightness. After the mortar has hardened, the joints are raked out in the ordinary manner and pointed, as in any masonry. These blocks were cast in wooden forms, the concrete for the entire structure being a 1-2-4

mixture. The exterior surface of the blocks is essentially composed of a 2 to 1 mortar. The forms were well made of smoothly-surfaced stuff, although no extraordinary care or skilled labor was used. Carpenters such as were available at Sandy Hill, and who had never done such work before, made the forms or moulds, and local workmen filled them. Although the surfaces of the blocks are not as smooth as could be obtained from metal-lined moulds, and although some are a little pitted, the results are excellent. The general effect of the structure, when observed as a whole, even close at hand, is that of masonry blocks of unusually smooth and regular texture. Experience with this structure shows what highly satisfactory results may be accomplished in this field of engineering construction, with any intelligent local labor when well directed. This is equivalent to stating that reinforced concrete work, either *en masse* or by the use of concrete blocks, is economically adapted to a wide range of construction throughout the country, where neither a high degree of skill in fabrication is needed nor more than the simplest appliances for mixing the concrete and putting it in place.

Although the railing has a most substantial and effective appearance, it will be found on examination to be composed of very simple members. The upright T-shaped pieces were cast in wooden moulds, which, after hardening, were put in place. The top rail was then cast in a suitable wooden form over their tops. Its appearance is in every way satisfactory.

Over each pier and the crown of each arch the roadway and sidewalk plates are cut throughout the entire width between the copings of the exterior spandrel walls. The interior spandrel walls are also cut at the same points. These openings form expansion joints, and are covered with 4 by ½-in. steel plates. No other provision is made for expansion or contraction throughout the entire structure. The ribs, piers and abutments form a continuous reinforced concrete mass a little more than 1 000 ft. long, without any provision for change of length, due to varying temperature or originating from other sources.

A complete system of drainage is provided in a simple way by frequent openings in the gutters through the deck of the bridge, permitting rain-water and melting snow to run directly into the river.



SECTIONS

OF BRIDGE OVER THE HUDSON RIVER AT SANDY HILL, N. Y. FOR THE COUNTIES OF SARATOGA AND WASHINGTON.

As shown on the plans, each rail of the railway track is securely fastened to a 10-in. channel, the flanges of which are embedded in the concrete. Suitably formed vitrified brick are laid on either side of each rail, and against these the asphaltic pavement abuts. The wearing surface of the pavement is brought up to such a height as to fall only a little short of being flush with the top of the rails, so as to make the entire roadway deck available for vehicular traffic when the railway track is not in use.

The cement used for this work was the well-known Ironclad brand, manufactured at Glens Falls, N. Y., about four miles from the site of the bridge. It was delivered and used under the standard cement specifications recommended by the Special Committee of the American Society of Civil Engineers on Uniform Tests of Cement. About 6 000 bbl. were required for the entire work.

The broken stone used was crushed local limestone, the specifications requiring 1½ in. as the largest dimension. The sand was also a local material. The concrete for the mass work of the bridge was all produced by a Smith mixer. The following paragraphs from the specifications show the character of the concrete used for all portions of the work:

"The concrete for all blocks, for the ribs and the interior spandrel walls above them, for floor and sidewalk plates, for the soffit sheets, and for the railings, and for all copings shall be formed of such portions of cement, sand and gravel or broken stone as will make a perfectly solid mass with a little surplus of cement over that required to fill the voids. The proportions of this mixture will be about equivalent to one measure of cement, two measures of sand, and four measures of gravel or broken stone.

"The concrete for the hearting of the piers, for the footing courses, and for other similar portions of the work, will be of such composition as to be about equivalent to one measure of cement, three measures of sand, and five measures of broken stone or gravel.

"In these provisions for the concrete it is the intention to take advantage of balanced sand, gravel or broken stone, *i. e.*, to use varying sizes for the purpose of decreasing the voids and using less cement to secure the best results. It will probably be necessary therefore to experiment with different proportions of varying sizes of sand and gravel or broken stone in order to ascertain the best mixture of the available materials at hand to secure the desired ends."

The concrete of the blocks was composed of the finer portions

of the crushed material, much of it—especially the exterior portions—being equivalent to about 2 to 1 mortar.

Work on the bridge was begun on May 4th, 1906, and all the concrete work was completed on January 8th, 1907, the total time occupied in construction being 8 months and 4 days.

The total cost of the structure, including the lamp-posts and the resident engineering organization, was about $77 000, about $5 000 of this amount being due to extra labor during nights and Sundays. This extra cost of over-time work was incurred to hasten completion, which was imperative in consequence of the dangerous condition of the old wooden bridge, the only available means of crossing the river at this point prior to the completion of the new structure. A little extra expense was also incurred in heating—by use of salamanders—various portions of the work during the extremely cold weather of the six weeks prior to the close of the work. Under ordinary conditions, therefore, the total cost of the bridge would scarcely have exceeded $72 000.

The method of casting the blocks with the internal openings, and the high degree of efficiency displayed in the prosecution of the work, were both due to the Resident Engineer, Mr. M. O. Kasson, who was continuously engaged on it from the beginning to the end of the construction.

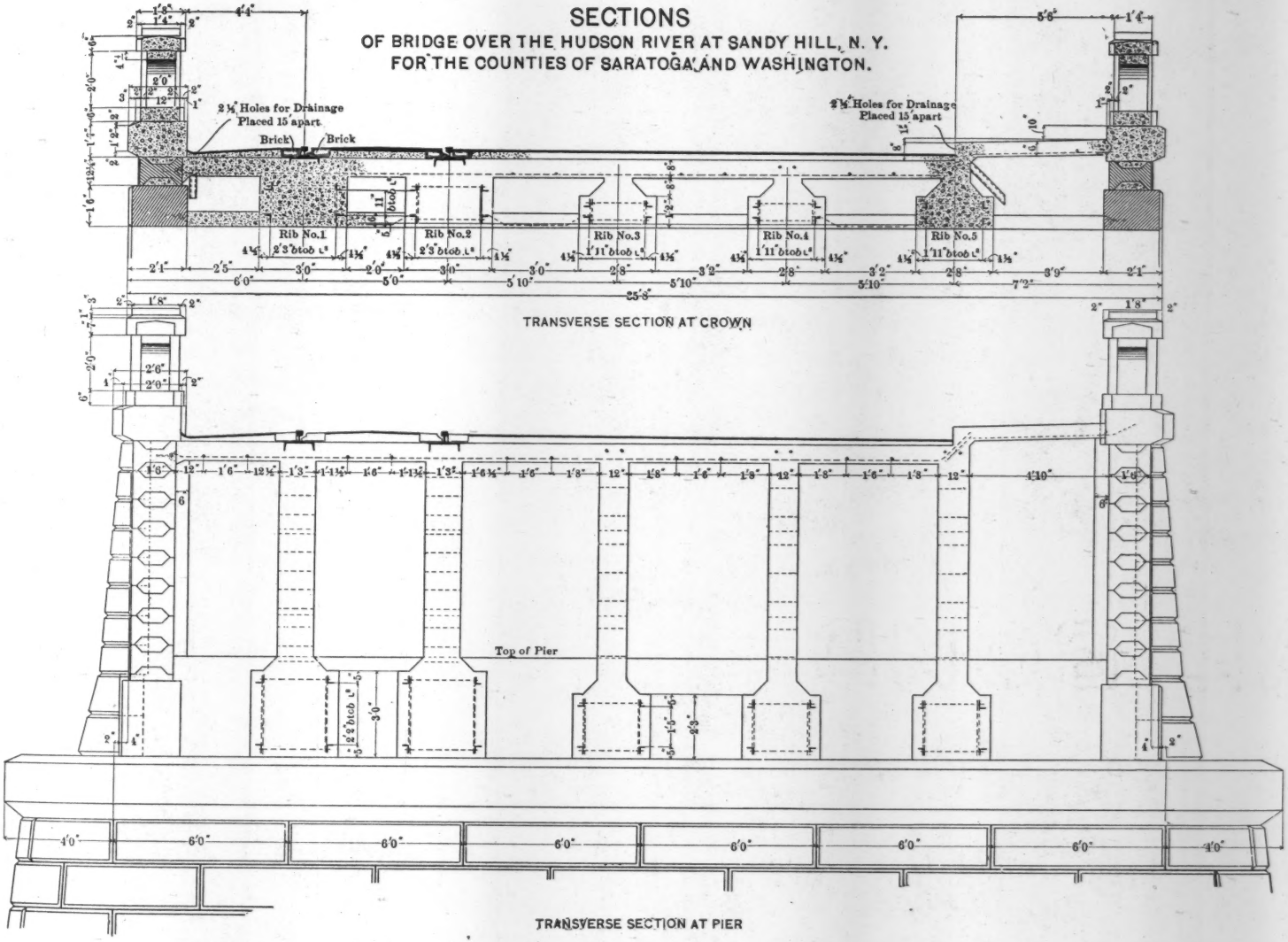
The entire work was done by day labor under the charge of the resident engineer.

The method of stiffening the ribs, it will be observed, is practically the Melan system of construction, and it was used under arrangement with the Concrete-Steel Engineering Company, of the City of New York, which company owns the Melan patents.

The river bed at Sandy Hill is practically clean rock with only a little detritus on its surface. During the summer the water is shallow, nowhere exceeding 2 or 3 ft. in depth. The foundation masses of concrete of all the piers, therefore, could be put in place quickly and cheaply within simple coffer-dams involving little time or cost. To this condition is due largely the comparatively short time required for the construction of the bridge and its low cost.

SECTIONS

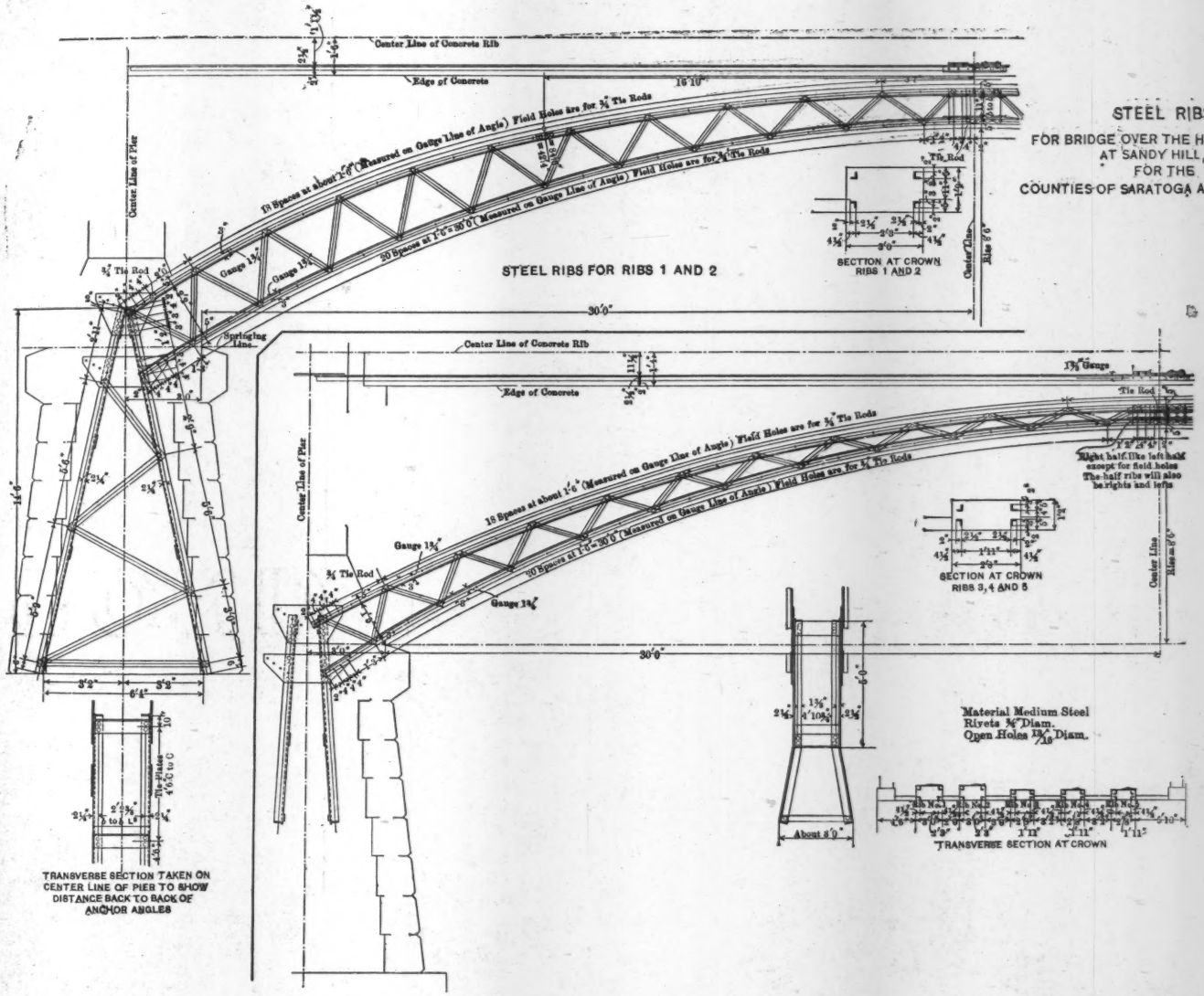
OF BRIDGE OVER THE HUDSON RIVER AT SANDY HILL, N. Y. FOR THE COUNTIES OF SARATOGA AND WASHINGTON.



TRANSVERSE SECTION AT CROWN

TRANSVERSE SECTION AT PIER

PLATE XX.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LIX, No. 1055.
BURR ON
REINFORCED CONCRETE BRIDGE.



DISCUSSION.

Mr. Quimby.

HENRY H. QUIMBY, M. AM. SOC. C. E. (by letter).—This bridge is an interesting illustration of the economic and æsthetic advantages of concrete as a material of construction. A steel-girder structure with paved deck would have cost more—the $72 000 would scarcely have paid for the superstructure alone—and the cost of proper maintenance capitalized would add a very considerable percentage to the total. Its appearance—its architecture—is also very pleasing, and the bridge will be a lasting delight to the community served by it.

The paper, while presenting the novel features and discussing the important points of the design, is somewhat meager in respect to several matters in which designers of concrete arches are especially interested. The stress diagram, and the method used in computing the stresses in the steel and concrete, including the temperature stresses, would add much to the value of the paper, as would also the details of the cost—the quantities and unit costs of the several divisions of the work.

The advantage of locality, of the proximity of the raw materials, may have been considerable, but, even then, the management must have been very efficient, for the cost of the work seems to have been quite low and the time consumed remarkably short.

The stresses in the arch due to changes of temperature will be high at the crown and at the spring, the whole effect of the change in length of arch, and the consequent rise and fall of the crown, being concentrated at these three points because the intermediate stretches are held rigidly to shape by the spandrel walls, unless the latter should be ruptured. The author's analysis of this effect would be valuable. The paper states that the bridge is designed to carry the specified loads without exceeding a compressive stress of 500 lb. per sq. in. in the concrete, and that, for all ordinary loadings, even with the prescribed railway loads, that compressive stress does not exceed about 400 lb. per sq. in. It is not clear whether the difference between the two represents the temperature stress, or whether the higher is merely the extreme fiber stress at the greatest eccentricity of the center of pressure.

The order in which the concrete was placed is an important consideration. If the soffit sheathing, in which was embedded the rods anchoring the block-built outer ribs to the inner ribs, was not constructed at the time of grouting the blocks, an opening between the blocks and the sheathing is possible, thus admitting water and subjecting the anchors to corrosion and ultimate destruction. The method of placing the rib concrete and that of the walls affects the question of initial stresses in the material, which, when com-

Mr. Quimby

bined with temperature stresses, are undoubtedly sometimes the cause of spalling and cracking.

The transverse reinforcement of the interior spandrel walls is novel, and will probably not be generally regarded as necessary. The cleavage that such reinforcement will resist is only longitudinal, either vertical as a spilt or diagonal as a shear of a column in compression, and such thin walls should not be subject to the former, and these are stressed so low vertically that the latter should not be feared. A very efficient reinforcement in the same direction would be flat stones embedded horizontally in the walls. This is found to add to the compressive strength of the concrete, and also to be economical, for the extra labor of placing is more than offset by the saving of cement concrete.

The channel sleepers under the track rails seem to be merely bedded on the concrete. If they are not bolted down firmly they will be likely to give trouble by loosening and pounding. It has been found necessary in this class of track construction to hold the rails down firmly to a sclid bearing on the concrete to prevent chattering, thus pulverizing the surface and corrugating the rails.

The wisdom of the plan of construction—block facing built up several courses and then grouted, and afterward filling in the heart with concrete which pretty certainly will shrink away from the facing—will not be generally conceded. Shrinkage openings, as well as the almost inevitable voids that the grout will fail to fill because of entrapped air and other stoppages of flow, will be likely to collect water, which is a destructive agent in all masonry; and the claim made in the paper that the construction is monolithic is hardly warranted. The economy of the method is open to question, which, however, should be answerable by the record of cost on this work compared with the fairly definitely known cost of wooden forms. The other expressed purpose of the adoption of this system of construction—to produce a satisfactory surface finish—could have been easily accomplished in monolithic work by removing the face forms within a day or two of placing the concrete and then scrubbing off the film from the surface, leaving the aggregate exposed to view. The result of this treatment is a face which is stone-like in texture and wearing quality, and not subject to the discoloration which mars a cement surface; or the face might have been tooled to remove the film, thus obtaining a surface which would be quite pleasing.

Mr. Slocum.

C. L. Slocum, Assoc. M. Am. Soc. C. E. (by letter).—The somewhat novel methods of design and construction used by the author, in this bridge, are of especial interest to the profession. The method followed in making each arch or series of arches independent and self-supporting, without leaving the falsework in

REINFORCED CONCRETE HIGHWAY BRIDGE OVER TRACKS OF THE NEW YORK, NEW HAVEN AND HARTFORD RAILROAD.

place until the completion of the abutments, or providing an abutment support or its equivalent to withstand the dead-load thrust of the completed arch spans, has many distinct advantages, particularly in places where of necessity only a portion of the bridge can be built at one time. This is especially true where a series of arches span railroad tracks in constant use, and where all the room desired is not to be had. Mr. Slocum.

Such conditions have been encountered in New Haven, Conn., in the construction of the arches of the reinforced concrete highway bridges of the New Haven Improvement of the New York, New Haven and Hartford Railroad. Here each highway bridge consists of four or five solid spandrel arches, each arch spanning a pair of tracks. In this improvement, in which the tracks are in a depression or cut, other recourse could not be had; there was not enough room to shift traffic from the site of the new structures, and besides, these highway bridges are so close to one another that certain spans in four of the five bridges of this class, have to be constructed to receive the tracks for traffic before the remaining spans can be completed.

The writer is of the opinion that, in most cases, fabricated units, or firm steel skeletons, are preferable to a less firm framework of rods and ties for reinforcement, especially in this and similar cases. In this particular construction, the fabricated units are made of old rails held together as rigid frames by old fish-plates and bolts, and bent-plates and bolts; longitudinal and transverse rods and vertical ties form a secondary meshwork between the built-up frames. In the design of this reinforcement, the writer, in addition to providing enough steel in the piers to take up the bending of the thrust at the springing line, giving an independent arch, has gone a step further. The frames of old rails are spaced so that, if it had been necessary, the usual kind of falsework, which is being used as a conservative measure, could be omitted, the lagging being hung from or attached firmly to the rail reinforcement. In other words, it is entirely feasible to erect these arches over tracks which are in constant use. Some European engineers, in design and construction, make the frame reinforcement large enough and strong enough to carry the requisite lagging and adjuncts. It is not at all improbable, in particular cases, that the saving in the cost of forms is greater than the increased cost of the added steel area required for self-support.

The general and detailed features of this construction have been described more fully in the technical press; however, by explaining the photograph, Plate XXI, perhaps what has already been stated will be better understood. The present main-line tracks at the extreme left of the photograph occupy the space necessary for the last

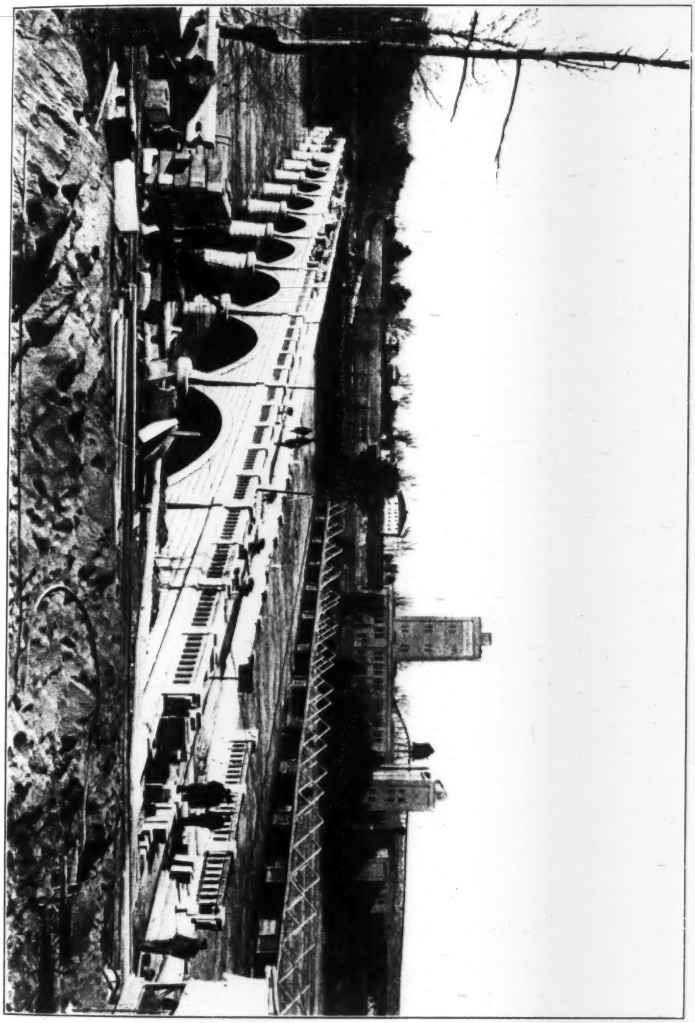
Mr. Slocum. arch and abutment. The arches are 31 ft. in the clear; the solid piers are plumb, and 2 ft. thick at the springing line. The last pier to which the remaining arch span is to be constructed is notched or cut out at the springing line to receive the last span. It is 12 in. thick at this point. The arch rail frames project out from this notch, and are connected to the vertical rail reinforcement which occupies the center of the pier. One side of this vertical rail reinforcement above the notch is barely covered with concrete, and the edges of the flanges of the rails are exposed in several places. There are frequent rabbets and other additional projecting rods of the meshwork to provide a good bond for the connection of the last arch. In addition to its own weight, this bridge, as shown in Plate XXI, has a temporary 7-ft. enclosed sidewalk which, during the noon hour, is quite congested with people from the nearby factories, etc. The main-line tracks are nearby, and although there is the usual tremor caused by trains passing frequently, there are no signs of weakness at this critical point, and none are expected.

Mr. Krellwitz. D. W. KRELLWITZ, JUN. AM. SOC. C. E.—Professor Burr has given a most interesting description of this bridge, which is remarkable for its block construction. The architecture rendered by such block construction is very good when one considers the small sum of money expended for beauty and strength combined. The details and sizes are of considerable interest, and show clearly a good design. The speaker would like to ask what the deflections were at the crown of each of the fifteen spans when the centering had been removed?

Mr Stern E. W. STERN, M. AM. SOC. C. E.—Would Professor Burr be so kind as to state how the thrust of one of the arches would be provided for, if the adjoining arch were removed?

Mr. Burr. WILLIAM H. BURR, M. AM. SOC. C. E. (by letter).—The methods of computation used in designing this structure were not set forth in the paper, as they involve only standard, and what may be called every-day, calculations in this particular field of work. As a matter of fact, computations were made on the assumption that the arch ribs are composed of blocks, and also that they constitute elastic arch members, *i. e.*, both the old and the new methods were used. The span is so short and the center lines of the ribs lie so near the line of resistance under either assumption that no steel reinforcement was required except as an extra measure to secure toughness and stiffness of the ribs.

Again, the shortness of the span and the solidity of the ribs, together with the monolithic bond between the interior ribs and the spandrel walls above them, give great uncertainty to the results of stress computations for variations of temperature. Furthermore,



REINFORCED CONCRETE BRIDGE ACROSS THE HUDSON RIVER, AT SANDY HILL, N. Y.

Mr. Burr.

it is essentially impossible to assign even approximate limits to temperature variations in such a mass of reinforced concrete protected by a floor deck. Although the temperature stresses thus become more a matter of judgment than computation, they were fully provided for.

The writer is strongly of the opinion that altogether too much has been made of temperature stresses in reinforced concrete work. This conclusion has been reached both by the aid of computations for such stresses and from extended observation of work of this class. Undoubtedly, such stresses exist, and cracks have frequently resulted from them. They must be recognized and allowed for, but there are various degrees of such recognition and a diversity of means for meeting the resulting conditions. A conservative judgment should be used in these operations rather than procedures which encourage divisions of the mass instead of retaining a monolithic character to the greatest possible extent.

As stated in the paper, each pier, with its steel reinforcement continuous with that in the arch ribs, has sufficient stability to hold either adjoining arch in position should the other be removed, although the latter event is so remote as to be essentially impossible.

The blocks of each arch ring were put in place before the adjoining mass concrete, so that the two parts became practically a unit by virtue of the steel clamps embedded jointly in them. The writer cannot share the opinion that the interior concrete mass of the piers can possibly shrink away from the enclosing blockwork, especially in view of the rigid bond formed between them by the steel clamps. Careful observations of the work at all points as it progressed showed that the thorough wetting which it received insured the essentially monolithic character of both the piers and the adjoining arches. There are doubtless limitations to the wise application of a block construction in this field of work, but experience with it in this bridge has been most gratifying as to economy and as to assuring solidity and real continuity of the masses in question.

The falsework was removed before the completion of the spandrel walls above the arch ribs, in all or nearly all cases, so that the exceedingly small settlement at the crown at that stage of construction had little significance.

The possible pulverizing of the cement under the channel base for the railway rails was fully considered, but, as the track is to be used for switching purposes only, it is believed that the means used to prevent such a result and to hold the track satisfactorily in place are amply sufficient to accomplish these ends.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 1056.

THE EFFECTS OF THE SAN FRANCISCO EARTHQUAKE OF APRIL 18TH, 1906, ON ENGINEERING CONSTRUCTIONS.*

REPORTS OF A GENERAL COMMITTEE AND OF SIX SPECIAL COMMITTEES OF THE SAN FRANCISCO ASSOCIATION OF MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS.

WITH DISCUSSION BY MESSRS. EDWIN DURYEA, JR., FRANKLIN RIFFLE, H. H. WADSWORTH, E. G. PERROT, LUTHER WAGONER, EUGENE W. STERN, J. K. FREITAG, W. W. HARTS, E. A. RIX, W. F. WHITAKER, A. M. BIENENFELD, ARTHUR L. ADAMS, S. G. HINDES, LOREN E. HUNT, B. BIENENFELD, J. P. CARLIN, CHARLES E. GOAD, R. B. GREEN, C. E. MOORE, A. L. A. HIMMELWRIGHT, LANGDON PEARSE, CHARLES DERLETH, JR., AND J. D. GALLOWAY.

REPORT OF GENERAL COMMITTEE.

EDWIN DURYEA, JR., *Chairman;* C. D. MARX, FRANKLIN RIFFLE, ARTHUR L. ADAMS AND WILLIAM W. HARTS.

Immediately after the earthquake of April 18th, 1906, and the destruction of San Francisco by the resulting fire, it was obvious that the assistance of skilled engineers could be of great value in planning the reconstruction of the city on new and better lines, and that valuable professional lessons would be learned by careful

*Presented at the meeting of May 1st, 1907.

study of the effects of the earthquake and fire on various engineering constructions. By virtue of its technical membership, the San Francisco Association of Members of the American Society of Civil Engineers seemed to be a fitting organization through which to proffer assistance to the municipal authorities, to study the effects of the earthquake and fire, and to collect technical information. Therefore, on April 28th, ten days after the earthquake, a meeting of the Association was held in Oakland to form an organization for these purposes.

At this meeting the work was outlined in the two general divisions: First, Plans for Reconstruction; and second, Effects of the Earthquake and Fire, as showing Needed Improvements in Structural Design. The Board of Direction of the Association ordered that an offer of the services of the Association be made to the Mayor of San Francisco, to assist him in any way desired. A number of committees were also appointed at that time to study the earthquake effects on various classes of structures.

In compliance with the Board's order, the aid of the Association was proffered to the Mayor through its President on the same afternoon, and, at the Mayor's suggestion, again in public at a meeting of the Committee of Sixty on Monday, April 30th.

Preparations for organizing the Committee of Forty on Reconstruction were already far advanced, however, and hence the assistance of the Association as an organization was not asked for by the Municipal Authorities. The Association, as a body, therefore, had no opportunity to assist in the plans for the reconstruction of the city. The Mayor, however, placed the following individual members on the Committee of Forty, as members of the sub-committees named below, so that, in effect, the Association took an important part in formulating such plans for reconstruction as were made at that time:

On the Sub-committee on Water Supply and Fire Protection.—Messrs. C. H. McKinstry, V. G. Bogue, A. M. Hunt, C. D. Marx, Edwin Duryea, Jr., and Marsden Manson.

On the Sub-committee on Reservoirs and Parks.—Mr. Marsden Manson.

*On the Sub-committee on Street-Widenings and Extensions.**—Messrs. V. G. Bogue, and Edwin Duryea, Jr.

* William Hood, M. Am. Soc. C. E. (not a member of the Association, or of the Committee of Forty), also joined in the work of this sub-committee, by invitation.

On the Sub-committee on Statistics.—Messrs. Marsden Manson, C. H. McKinstry, V. G. Bogue, and Edwin Duryea, Jr.

On the Revision of Building Laws.—Mr. J. D. Galloway.

On the Committee of Sixty.—In the first few days after the fire, very important aid was rendered to the city by two members of the Association, Messrs. George L. Dillman and A. M. Hunt, the former having been in charge of the emergency restoration of the water-supply, and the latter of the emergency restoration of lighting and transportation.

The Association, not being needed to aid the city as a body, then devoted its whole attention to the second phase of the subject: A study of the earthquake effects, as showing needed improvements in the design of engineering works. At the first meeting, April 28th, several committees were appointed by the Board of Direction for this purpose. The personnel of these committees, as afterward completed, is given at the beginning of the report of each committee.

Through these committees, the Association has studied the physical effects of the earthquake and fire, with the results shown in the accompanying committee reports. It is believed that the data thus gathered and the conclusions deduced therefrom are of permanent benefit to the profession, and therefore they are presented for publication in the records of the Society. It is believed that after publication there will be a full discussion of the several subjects by the members of the Association in San Francisco, as well as by the Society. As far as practicable, all questions dealing with the earthquake effects will be answered by members of the Association who have had special opportunities for observing the facts.

Before closing the report of the General Committee, it seems proper to give a brief abstract of the conclusions reached by each committee.

It must be understood that the Association, as a body, is not responsible for the conclusions reached by the several Committees, and that the General Committee does not necessarily concur in all of them. Differences of opinion are sure to exist, and will no doubt be brought out fully in the discussion.

COMMITTEE ON GEOLOGY.*

(1)—"The earthquake of April 18th, 1906, was due to the relief of accumulating stresses in the earth's crust, by a movement along a previously known fault-plane."

(2)—"While the time of occurrence of such movements cannot be predicted, the relief of the accumulated stresses gives reasonable assurance of freedom (within the area affected) from a recurrence of earthquakes of similar intensity, for some time—probably at least fifty years."

(3)—"By reason of the existence of this and many other fault-lines in California, through the agency of which stresses may be relieved when they reach the danger-point, earthquakes of greater intensity than the recent one are hardly to be expected, even in the remote future."

(4)—"The disastrous results following the recent earthquake * * * were largely due to a lack of safe construction, as ordinarily understood; and, if the lessons which may be drawn from it are followed in the future, it is believed that the Pacific Coast can be made safe as a habitation and field of investment, as far as earthquakes may affect conditions."

COMMITTEE ON BUILDINGS.

(1)—*Earthquake Effects.*—"Sufficient evidence is at hand to warrant the statement that a building designed with a proper system of bracing to withstand wind at a pressure of 30 lb. per sq. ft. will resist safely the stresses caused by a shock of an intensity equal to that of the recent earthquake."

"The prime requisite of the structure is elasticity. * * * To this requirement, the building with a timber or steel frame answers very well. The reinforced concrete structure does so also, with * * * exceptions * * *. The building with stone, brick or block construction, having horizontal mortar joints, does not answer the requirement at all. It may be stated, as one of the most obvious lessons of the earthquake, that brick walls, or walls of brick faced with stone, when without an interior frame of steel, are hopelessly inadequate. As a method of building in earthquake countries, such types are completely discredited."

"Foundations did not suffer at all, no instances of any damage having come to hand. * * * The evidence is that foundations well built, along accepted lines, are adequate."

"It may be questioned whether difference in workmanship was not responsible for many of the results. While it is true that good workmanship gave better results than ordinary, it is still the

* The personnel of this and other Committees mentioned in this report is given with the reports of each Committee, printed in the following pages.

opinion of the writers that it was mainly a question of design. * * * Speaking generally, buildings of brick walls and wooden interiors cannot be built which will not be wrecked in a severe shock, it being a fault of design and not of materials or workmanship."

(2)—*Damage by the Fire.*—"All materials were destroyed when directly exposed to the fire for any length of time. Brick, terra cotta, granite, marble, sandstone, steel, cast iron, concrete, plaster, cement, timber—all failed. Especial emphasis is placed on this point. Anyone with an interest in any one material can find evidence to support a claim for it; yet, impartial observation leads to the conclusion that, when a material was subjected to what would be considered a test fire, it was destroyed."

"No foundations were found injured by the fire."

"The steel frames were the least injured of any part of the various structures. Where properly protected, there was no injury. Where the protection was faulty, or where there was none, the destruction was complete."

"A logical deduction from the statement that all materials were destroyed is the conclusion that all structural parts of a building, of whatever material constructed, must be protected by another material which will be a more or less complete loss in a fire. This applies to a steel frame, to floors of any type, and to roofs. It is impossible to protect some parts, such as fronts, partitions, and other parts directly exposed."

"Any deductions from the fire must be those based upon a general conflagration, and not those of an isolated fire. In view of the complete destruction of all materials, it becomes a question as to what should be done to make a building fire-proof. * * * The only statement that can be offered is that the best insurance for buildings would be the isolation of a district containing nothing but fire-proof structures. A general conflagration would then be impossible."

COMMITTEE ON WATER-WORKS.

"*First.*—That greater attention in the future should be given to the avoidance, when possible, of threatening geologic faults, in the location of important water-works structures, than has sometimes been given in the past."

"*Second.*—That skillfully designed and well-built earthen dams are structures proven by the great earthquake of April 18th, 1906, to be of great stability, and deserving of increased confidence."

"*Third.*—That concrete dams of gravity section are able to withstand earthquake shocks of great severity without damage."

"*Fourth.*—That distributing reservoirs, pumping machinery, elevated tanks and stand-pipes, if made secure as to foundations,

and intelligently designed along lines of the best accepted practice, will be found capable of withstanding earthquake shocks sufficiently severe to work the general destruction of most buildings."

"*Fifth.*—That pipe lines or conduits, of any character whatever, are almost certain to fail when intersected by a plane of large movement, whether that movement be in the nature of a faulting, or of a sliding or settling of the earth in consequence of earthquake shock; and that, in choosing locations for such structures, it is prudent, as far as practicable, to avoid localities subject to such movements."

"That in planning distributing systems for cities, the important supply pipes should be located along routes carefully selected for stability; and that areas liable to serious disturbance should be so segregated in the piping system as to make them easily and quickly separable from the remainder of the system."

"*Sixth.*—That close adherence should be given to the recognized standards of the best present practice, such as the use in distributing systems of an abundance of suitably placed gates for isolating different pipes and different sections; the preservation of such gates in working condition; and having their location easily ascertainable, both in the street and from the office records; the duplication, along widely separated routes, of the critically important main supply pipes; the maintenance in as close proximity as may be to the centers of use of an ample storage supply of water; and the use of types of substantial design in working out plans for structures subject to possible earthquake shock."

COMMITTEE ON LIGHTING AND STREET-RAILWAY TRANSPORTATION.

"The injury caused to the street-car tracks was relatively slight," and only "on ground that had been filled."

"The slots of cable roads * * * (with few exceptions) were found closed * * * only in the burned district, and the evidence points to its being due mainly to expansion caused by the excessive heat. The fact that the Geary Street Line, which is a cable road, operated for several hours after the earthquake, is very pertinent." The cable roads, however, were put entirely out of use by the fire and the falling debris.

"Many of the trolley lines outside of the burned district * * * could have been operated at any time," except for "the fire-hazard from the tangle of other wires. The trolley lines unquestionably substantiate the claim that such a system is flexible, and admits of emergency work to an extent not equalled by any other."

The street-railroad power-houses were but little injured except by the falling of a brick stack and the collapse of a roof. "Brick stacks * * * were in most instances broken off, while well-guyed steel stacks stood without injury."

With one exception, "the various gas-works were uninjured. The damage to mains was principally due to explosions. As service was gradually restored, it was found that the house pipes, except in very few instances, were not damaged."

Electric lighting plants were but little injured by the earthquake, but the fire destroyed many sub-stations and overhead distributing lines. "The underground conduit system and the Edison tubing came through both earthquake and fire in very good shape."

Committee on Sewers.*

No report has as yet been made by this committee, it is said, because the injury to sewers cannot be known conclusively until after the first heavy rain. From a letter† written by H. D. Connick, one of the members of the committee, and the Assistant Engineer in charge of San Francisco's sewers, the following information is condensed:

In the rocky portions of San Francisco the sewers were not affected. In portions where the rock was overlaid with sand, there were no permanent displacements except where the original ground supported a fill; in such areas settlements occurred, and the sewers were destroyed. In filled-in tidal areas, marsh-lands and swamps there was considerable movement in a number of places (the greatest near 16th St. and Valencia St., where the settlement was 5 ft. and the lateral movement 6 ft.) and in all such disturbed areas the sewers were destroyed.

All the masonry sewers injured, with one exception, were constructed, a number of years ago, of brick laid in a poor quality of lime mortar—in fact, in many old sewers there is nothing but sand in the joints of the brickwork.

In San Francisco a reinforced concrete sewer, on a pile foundation, where the ground does not furnish suitable support, will probably give the best satisfaction.

It is believed that $100 000 will cover the total damage to the San Francisco sewers by the earthquake.

Committee on Railway Structures.

"The structures peculiar to railroads which were affected, are embankments, trestles, bridges, tunnels, and water and oil tanks."

Embankments.—These settled more or less where they crossed marshes or were underlaid by soft strata. In one instance (across a tule marsh) the settlement was as much as 11 ft., while in another case (miles from the fault) a settlement of only 6 in. was accompanied by a horizontal displacement of 3 ft.

Trestles.—The damage to trestles was small, except on the North

*This Committee consisted of Messrs. Marsden Manson, H. Kower, and H. D. Connick.
†*Engineering News*, September 20th, 1906.

Shore Railroad, where a trestle of framed bents on piles, 600 ft. long and 70 ft. high, was thrown down, and portions of another trestle were thrown entirely off the piles, the piles themselves being moved down stream. These trestles were across soft ground, and near the fault-line.

Bridges.—"Draw-bridges * * * around San Francisco Bay, being generally on soft ground, were affected by a slight movement of their piers, in many cases resulting in the bridge binding so that it could not be opened until some repairs were made."

"A steel draw-bridge, over Petaluma Creek, was open at the time of the earthquake, and was thrown off its center 2 ft. to the east and 1 ft. to the north. It was a 220-ft. span, and on four iron cylinders, filled with concrete, on pile foundations."

Fixed spans, with a few exceptions, were not affected seriously. Where affected it was by a movement of the piers or abutments relatively to each other.

Tunnels.—"In general, tunnels seem to have been affected only by the displacement or loosening of the material in the sides and roof, caused by the shaking of the ground. The effect of this was to crush the timbers."

"The new tunnels being constructed by the Southern Pacific Railroad on the Bay Shore line, in the southern part of San Francisco, were not injured at all."

Tanks.—"Several tanks on the different roads were thrown down. In all cases, however, these were on wooden or cast-iron frames, not braced to withstand the twisting motion which accompanied this earthquake. Tanks on riveted frames did not fall, though in several instances they were severely damaged and thrown out of plumb."

Overturned Cars, etc.—"At Chittenden Station, * * * three standing box-cars, two of them loaded, were turned over."

"A narrow-gauge passenger train, consisting of engine and three cars, standing on the track at Point Reyes, was thrown over on its side."

"A freight-train, running near Chittenden Station, was derailed."

Committee on Highway Structures.

No final report has as yet been received from this committee. As far as ascertained, the earthquake did very little damage to highway bridges.

The reports in full of the several committees appointed as stated in this report will be found in the Appendix. It is hoped they will bring out a full discussion from engineers who are interested in the effects of earthquakes on engineering structures.

APPENDIX A.

REPORT OF COMMITTEE ON THE GEOLOGY OF THE EARTHQUAKE.

COMMITTEE:
H. L. HAEHL, OTTO VON GELDERN, LUTHER WAGONER, C. E. GILMAN.

At 5.12 A. M. on April 18th, 1906, a violent earthquake shock was felt throughout the western portion of Central California. The shock lasted about one minute, and it has been followed, to date (August, 1906), by a series of lesser shocks gradually diminishing in intensity and frequency. Much damage was done to buildings by the first shock (particularly in San Francisco, Santa Rosa, San José, and Stanford University), and several hundred lives were lost. Disastrous fires broke out in San Francisco and in Santa Rosa, and the damage from earthquake was multiplied many times by their ravages.

The damage to buildings, while large, was confined to those faulty either in design or in construction. Carelessness in fulfilling contracts, and inadequate building inspection, are responsible for most of the loss by earthquake. Aside from the injuries to, and by reason of, the structures of Man, there is nothing to indicate a calamity, except, perhaps, on the fault line, to be hereafter described. The face of the earth is unchanged, and neither vegetable nor animal life has suffered. The results, so disastrous to buildings and engineering structures, can and will be provided against in the future, and a study of the geological elements of the disturbance reveals no reason why California need expect earthquakes of such intensity as to render structures dangerous, if properly designed and honestly built. Luckily, the cause of the earthquake is now well known, and was, indeed, immediately suspected by all geologists familiar with the geology of the Coast region.

It is known that the earth's crust has been and is now being distorted, fractured and displaced; but geologists deal with a unit of time which is beyond the ordinary comprehension, and it is only occasionally that an acceleration of these movements brings them fairly before the lay mind, and calls attention to the fact that, while we have been considering the earth as something fixed and stable, stability is only a relative term.

Many theories have been advanced to explain the phenomena of earth movement; in fact, the causes themselves are probably many in number, complex in character, and but imperfectly under-

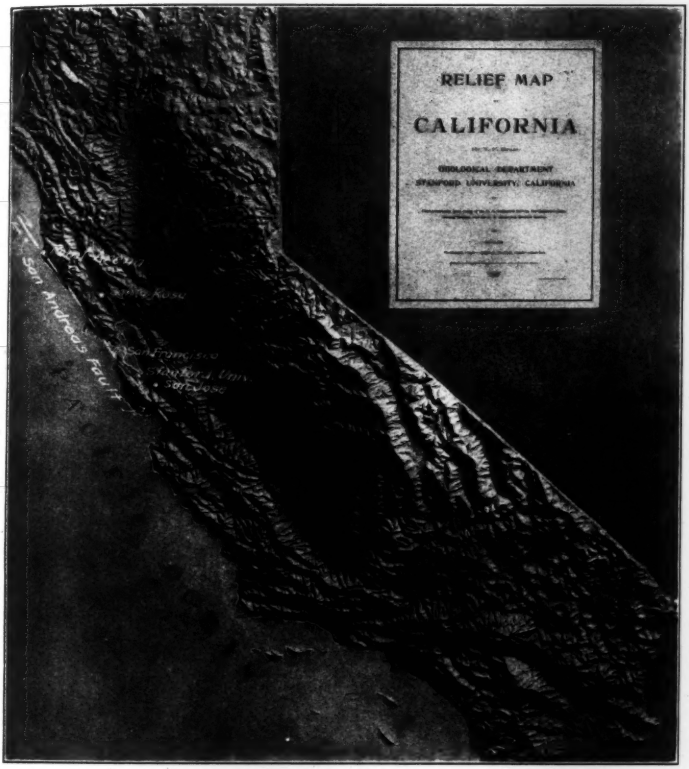


FIG. 1.—RELIEF MAP OF CALIFORNIA.

FIG. 2.—OFFSET OF 8 FT. IN FENCE LINE IN PORTOLA VALLEY.

APPENDIX A.

REPORT OF COMMITTEE ON THE GEOLOGY OF THE EARTHQUAKE.

COMMITTEE:
H. L. HAEHL, OTTO VON GELDERN, LUTHER WAGONER, C. E. GILMAN.

At 5.12 A. M. on April 18th, 1906, a violent earthquake shock was felt throughout the western portion of Central California. The shock lasted about one minute, and it has been followed, to date (August, 1906), by a series of lesser shocks gradually diminishing in intensity and frequency. Much damage was done to buildings by the first shock (particularly in San Francisco, Santa Rosa, San José, and Stanford University), and several hundred lives were lost. Disastrous fires broke out in San Francisco and in Santa Rosa, and the damage from earthquake was multiplied many times by their ravages.

The damage to buildings, while large, was confined to those faulty either in design or in construction. Carelessness in fulfilling contracts, and inadequate building inspection, are responsible for most of the loss by earthquake. Aside from the injuries to, and by reason of, the structures of Man, there is nothing to indicate a calamity, except, perhaps, on the fault line, to be hereafter described. The face of the earth is unchanged, and neither vegetable nor animal life has suffered. The results, so disastrous to buildings and engineering structures, can and will be provided against in the future, and a study of the geological elements of the disturbance reveals no reason why California need expect earthquakes of such intensity as to render structures dangerous, if properly designed and honestly built. Luckily, the cause of the earthquake is now well known, and was, indeed, immediately suspected by all geologists familiar with the geology of the Coast region.

It is known that the earth's crust has been and is now being distorted, fractured and displaced; but geologists deal with a unit of time which is beyond the ordinary comprehension, and it is only occasionally that an acceleration of these movements brings them fairly before the lay mind, and calls attention to the fact that, while we have been considering the earth as something fixed and stable, stability is only a relative term.

Many theories have been advanced to explain the phenomena of earth movement; in fact, the causes themselves are probably many in number, complex in character, and but imperfectly under-

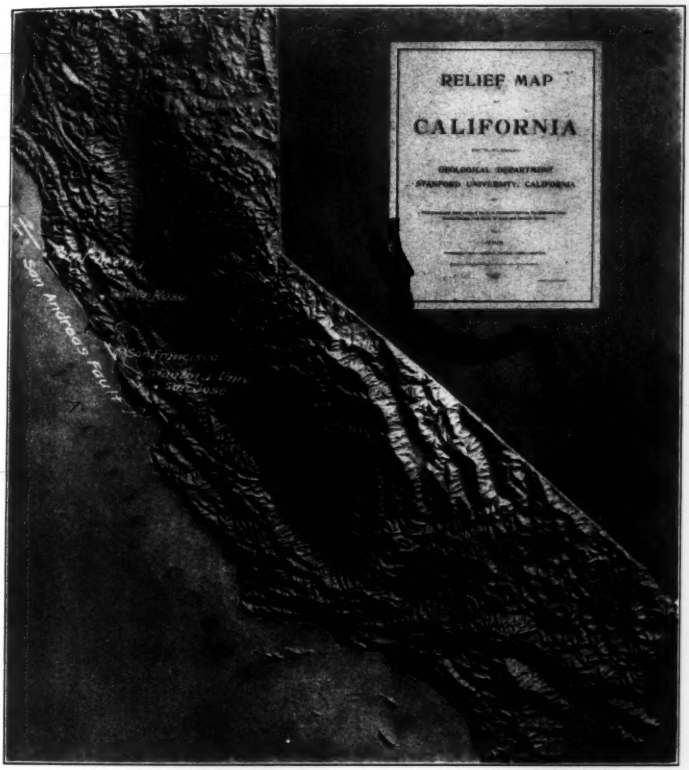


FIG. 1.—RELIEF MAP OF CALIFORNIA.

FIG. 2.—OFFSET OF 8 FT. IN FENCE LINE IN PORTOLA VALLEY.

stood. Movements not accompanied by volcanic phenomena (tectonic) are believed by some to be due to the contraction of the earth's crust in cooling, while others hold that they are due to the expansion of the surface by reason of heat rising from within. The transportation of enormous quantities of material from one portion of the earth's crust to another by the denuding and depositing action of streams is also believed gradually to set up strains in the earth's strata which in the end cause fracture and movement. The idea that barometric pressure and the moon's phases are causes of earth fractures and the resulting earthquakes has been abandoned; yet scientists feel that tidal strains due to the attraction of terrestrial bodies may at times at least assist in bringing about earth movements.

Whatever the causes, it is certainly clear that in the earth's crust stresses exist which accumulate to the point of breaking the strata and causing distinct movements, sometimes slow and uniform, and sometimes rapid and spasmodic. The more rapid movements are classed under three heads:

First.—Volcanic disturbances, explosive in character, due probably to the explosion or condensation of gases which accumulate beneath the earth crust, and confined to areas of volcanic activity.

Second.—The breaking of strata under stresses. Evidences of this are common. Beds originally deposited in horizontal layers are found up-tilted at all angles, and sometimes completely reversed in sharp, closely pressed folds.

Third.—Rock masses slipping one against another, either horizontally or vertically, along planes of fracture or faulting. These fault planes extend into the earth to unknown depths, and the lines of faulting can often be traced for hundreds of miles along its surface. They are the safety valves of their particular localities—the lines of weakness, or of least resistance, along which the accumulating strains of the earth's crust are relieved.

It was a horizontal movement of this last type, along a well-known fault line, which set up the vibrations of the earth's crust which constituted the earthquake of April 18th. It should be borne in mind that there is a marked distinction between an earth movement, as here referred to, and an earthquake. They are cause and effect. The recent earth movement along the particular fault line responsible for the location of the earthquake has been observed to be as great as 10 to 20 ft., measured by offsets in fences and roads, yet the amplitude of the resulting earthquake at Mount Hamilton, 13 miles east of San José, recorded by the seismographic apparatus of the Lick Observatory, was only 2.03 in. The earth movement is believed to have been felt in San Francisco and Oakland as a gentle shifting to the southeast, probably coincident with

the destructive movements, and, by reason of its amplitude, was not recorded by instruments. It is evident that the movement of a rock mass many miles in length, slipping for 10 or 20 ft. along a great fracture, would cause violent tremors to pass through the earth for many miles, diminishing in intensity as they traveled away from the fault line. It must also be apparent that such a movement may be followed by a series of minor movements and adjustments of the fractured zone, each causing its smaller earthquake, and that these might be expected to last for weeks or months and to diminish in intensity and frequency until a state of relative equilibrium was again established. The past history of earthquakes records this decreasing scale of intensity of shocks, and, while it may not be laid down as an exact law, the experience has been so general that it may be accepted as one, within certain limits.

An examination of the relief map of California, Fig. 1, Plate XXIII, reveals the fact that the State is traversed by many important valleys having axes which are approximately parallel. As might be expected, geological study shows these to be fault valleys, the troughs of which are originally due to fracture and displacement of the earth's crust, rather than to the cutting action of streams. The entire western coastal region of the continent is characterized by these fault lines. They are directly responsible for the number of earthquakes in these localities, in that they give way and relieve the stresses in the earth's crust. Earthquakes, therefore, are frequent, but in the main not intense. It is certainly open to question if such a region is not preferable to those that have no such safety valve, and whose disturbances, while rare, may be too intense to be provided against successfully. Destructive earthquakes of greater intensity have certainly occurred in other parts of the United States, less populous, and without the publicity given by an unprecedented fire.

The San Andreas-Stevens Creek fault line, along which the movement took place in this instance, has been indicated by a line on the relief map (Fig. 1, Plate XXIII), and also upon the map, Fig. 1. The directions of the relative lateral movement are shown by the arrows. The vertical movement, if any occurred, was slight. The geological section, *AB*, Fig. 2, shows in a general way the geological structure along a section through the Coast Ranges and Santa Clara Valley near San José, indicated and lettered *AB* on Fig. 1. The fault line crosses the section at Stevens Creek. It will be noted that San José stands upon a deep alluvial deposit, in which borings 1100 ft. deep have not encountered bed-rock. The unstable character of these deposits is doubtless responsible for the unusual extent of the damage done by the earthquake in San José.

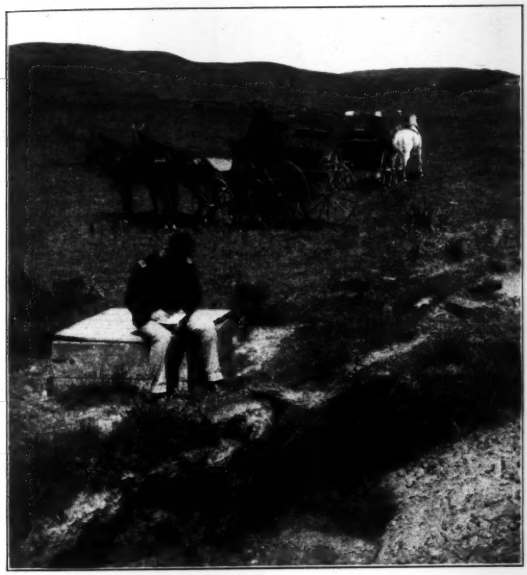


FIG. 1.—FURROW ALONG FAULT LINE NEAR SAN ANDREAS RESERVOIR.

FIG. 2.—DISPLACEMENT OF SOIL IN VALLEY OF PAJARO RIVER. THIS IS NOT ON THE FAULT LINE, BUT SHOWS THE CHARACTER OF SETTLEMENTS IN ALLUVIAL DEPOSITS.

The section shown is somewhat typical of the structure adjacent to the fault line throughout the region south of San Francisco. There are fault lines also on either side of the Stevens Creek fault, but, as far as known, no movement recently occurred along them. The fault line indicated on the eastern side of the valley, however, is probably one along which movement occurred in 1868 when a shock of some importance, but lacking much of being equal in intensity to that of April 18th, did some damage.

The fault block lying between the fault along the eastern side of Santa Clara Valley and the San Andreas-Stevens Creek fault has undergone a series of movements since Miocene time, as a part of the geologically active Coast Ranges. It has also experienced an uplift on the western side, with a coincident downthrow on the eastern side, at some time since the Pliocene, as is evidenced in this section by the up-tilted Pliocene gravels making up the hillsides on the west side of the Santa Clara Valley. The San Andreas-Stevens Creek fault line has been subjected to even more recent movements; it was therefore poorly cemented, and was the first to give way under the accumulating strain.

Map A (Fig. 1) shows the location of the fault line from Bodega Head to a point in the Coast Ranges west of San José. The region near Olema, shown on this portion of the fault line, is believed to have sustained the greatest movement. The greatest damage done was adjacent to the portion of the fault line within the limits of this map. The rift has been traced in detail from near Point Arena, in Mendocino County, on the north, to near San Juan, in San Benito County, on the south—about 185 miles. Topographic features and scattering reports of fractures observed indicate that the fault line probably extends beyond these limits, both northward and southward, and further investigation may develop connected evidences of movement on these extensions. However this may be, the already well-defined rift is of sufficient length to indicate a profound fracture which must extend to great depths below the earth's crust.

It will be noted on the accompanying maps that the line of the fault is characterized by a series of long, narrow valleys, comparatively straight sections of creek beds, and saddles in mountain ridges. The scarps, sink holes and troughs peculiar to fault lines along which recent movements (geologically speaking) have occurred are all in evidence along this line, and had been recognized and studied in some detail by the geologists of the California universities. After the earthquake, a movement was at once suspected along this well-known line, and subsequent investigation resulted in a verification of the work of location previously made. The exact location of the line of fracture is made evident by a

SKELETON MAP OF A PORTION OF ZONE OF
CALIFORNIA EARTHQUAKE OF APRIL 18TH, 1906.
Showing topographic features along fault line.

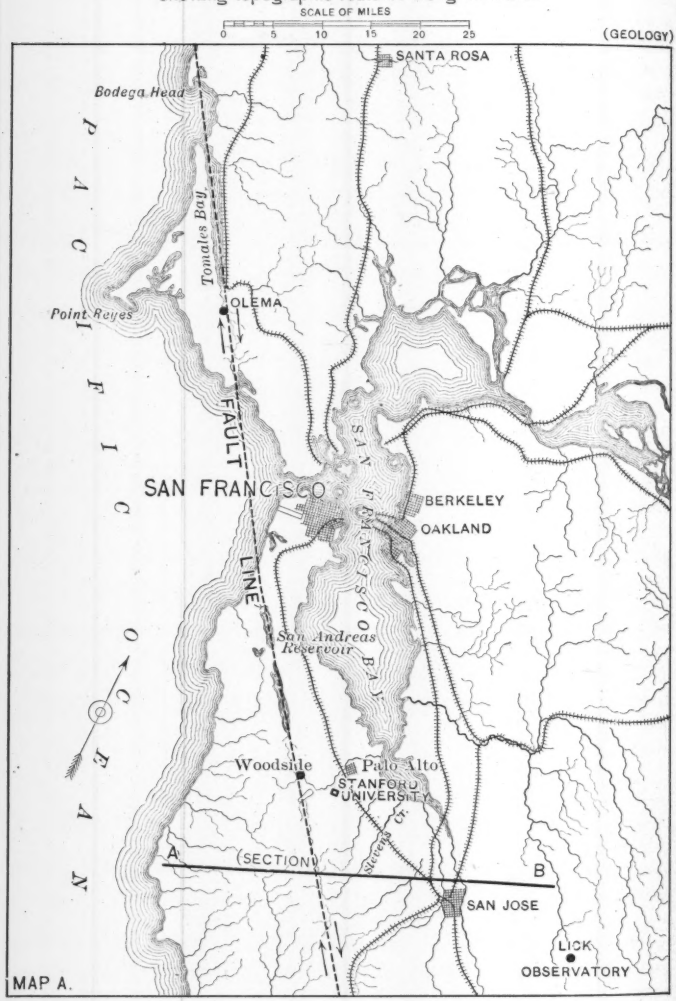


FIG. 1.

furrow several feet in width across the open fields and bare hills. The furrow takes on the nature of plowed ground, with occasional cracks, both longitudinal and transverse, and is remarkably straight. Through the forests the line is traceable by trees uprooted, shattered, or leaning. Everywhere, roads, fences and conduits are offset or telescoped. The accompanying photographs on Plates XXIII and XXIV give some indication of the character of the fault line phenomena.

In conclusion, it may be stated:

That the California earthquake of April 18th was due to the relief of accumulating stresses in the earth's crust, by a movement

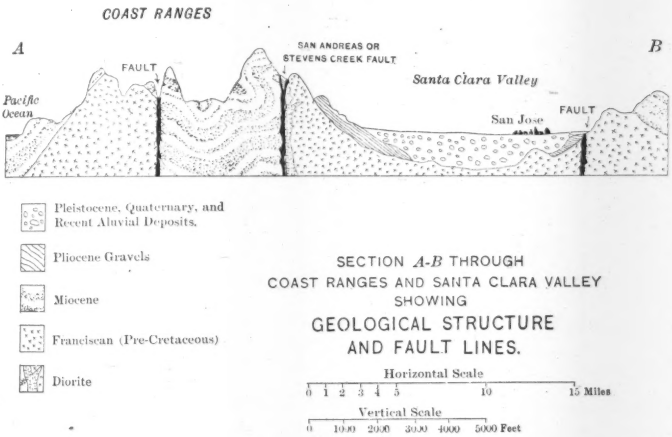


FIG. 2.

along a previously known fault-plane whose fracture, by reason of geologically recent movement, offered a line of least resistance;

That, while the time of occurrence of such movements cannot be predicted, the relief of the accumulated stresses gives reasonable assurance of freedom from a recurrence of earthquakes of similar intensity, within the area affected, for some time—probably at least fifty years;

That, by reason of the existence of this and many other fault-lines in California, through the agency of which stresses may be relieved when they reach the danger-point, earthquakes of greater intensity than the recent one are hardly to be expected, even in the remote future;

That the disastrous results following the recent earthquake, both as to loss of life and property, were largely due to a lack of safe construction, as ordinarily understood; and, if the lessons which may be drawn from it are followed in the future, it is believed that the Pacific Coast can be made safe as a habitation and field of investment, as far as earthquakes may affect conditions.

APPENDIX B.

REPORT OF COMMITTEE ON FIRE AND EARTHQUAKE DAMAGE TO BUILDINGS.

COMMITTEE:
J. D. GALLOWAY, M. C. COUCHOT, C. H. SNYDER,
CHARLES DERLETH, JR., C. B. WING.

The earthquake in the western portion of California, and the resulting fires in San Francisco and Santa Rosa on April 18th, 1906, caused a general destruction of engineering structures which is of interest to many engineers.

It may be stated that this report has been delayed in order that as complete data as possible might be obtained. The damage by fire and earthquake was extensive; and, in the case of fire damage, the effect on concealed parts was hidden, and could only be ascertained when the buildings were stripped. This can hardly be said to be complete as yet, but it was felt that the report could not be delayed any longer for that reason.

Some members of the committee have been employed in a professional capacity in the examination of damaged buildings. These statements, when considered in connection with the press of business in an extraordinary period, must be taken as the excuse for not furnishing to the profession an earlier report.

The region affected by the destructive effects of the shock was some 300 miles long by 50 miles broad, and covered the most populous part of the State, including the cities of Santa Cruz, San José, Alameda, Oakland, Berkeley, San Francisco, Napa and Santa Rosa, besides a number of smaller towns. No attempt is made here to enter into a detailed study of the shock, or its cause, such topics being treated by the Committee on Geology.

The shock was followed by destructive fires in San Francisco and Santa Rosa, which destroyed the entire business districts of both places. The area burned over in San Francisco slightly exceeded 4 sq. miles. Santa Rosa had a population of about 8 000, and its entire business district was burned. A number of institutions, such as Stanford University at Palo Alto, the State Hospital for the Insane at Agnews, and St. Patrick's Academy at Menlo Park, were badly wrecked.

No statistics are at hand as to the number of people killed, but it might reach 750. At Agnews' Asylum 112 people were killed, of whom more than 100 were patients.

It is not believed possible to locate any particular place as the center of disturbance, the effects being about the same in all parts of the area named. The character of the ground had considerable to do with the destructive effects, buildings on rocky hills suffering much less than those on alluvial plains.

Any consideration of the damage to buildings must be preceded by a description of the type of buildings; hence this report is naturally divided into three parts, namely:

I.—Description of Buildings;
II.—The Earthquake Damage;
III.—The Fire Damage.

I.—Description of Buildings.

For the purposes of this report, buildings are divided into five types, as follows:

Type 1.—Wood frame structures;
Type 2.—Structures with brick, stone or concrete block walls and timber floors and partitions;
Type 3.—Structures with brick and stone bearing walls, metal interior frames, and fire-proof floors;
Type 4.—Structures with steel frames supporting all wall and floor loads;
Type 5.—Reinforced concrete structures.

It must be recognized, of course, that there were various combinations of these types, but the classification is sufficiently definite for present purposes. Types 3 and 4 are nearly the same, but both fire and earthquake damage differed considerably in the two.

Building methods were about the same as in other parts of the United States, and hence a brief description is deemed sufficient. It must also be recognized that here, as elsewhere, there was much difference in structures placed under the same heading, the well-braced steel-frame building standing next to one without any attempt to supply bracing of any kind. For obvious reasons, no special building can be designated as being thus defective.

Type 1.—Wood Frame Structures.—With a few exceptions, all residences, including 2-story and 3-story flats, in the territory named, were of timber. The San Francisco ordinance allowed them to be built 45 ft. high. Some higher buildings existed in San Francisco, but no data are at hand regarding earthquake damage, as they were burned. The construction was generally with studding one story in height, the succeeding story studding resting upon the floor joists. Some balloon framing was used, but it was not general. This type included residences, and, in some small towns

FIG. 1.—EARTHQUAKE DAMAGE TO COLONNADES, STANFORD UNIVERSITY. NOTE CONSTRUCTION OF SPANDRELS.

FIG. 2.—EARTHQUAKE DAMAGE TO LIBRARY OF STANFORD UNIVERSITY.

and outside the fire limits in the larger towns, many store buildings were built in this way.

Type 2.—Masonry Walls, Timber Floors and Partitions.—This type of structure formed the more pretentious buildings of the cities and towns outside of San Francisco, and also the buildings at Stanford University, Agnews' Asylum, St. Patrick's Academy, and County Court Houses. In San Francisco, the great majority of business blocks were of this type, as were also the large private hotels and apartment houses. Under the San Francisco building ordinance, they could be built 100 ft. high, or 8 stories. Buildings of 2 or 3 stories might have, with the brick walls, a system of timber columns and girders, or a partition of timber studding, carrying the floors and roof. In higher buildings, cast-iron or steel columns and steel girders were used to support the joists. Roofs were of timber framing, covered with tin, or tar and gravel.

The Stanford University buildings had walls of brick with a facing of sandstone, one general type ruling throughout. Some of the older buildings had full stone walls, and these were but slightly injured. They were built by various architects. In some, wooden roofs without trusses were used; in others, steel or wooden trusses; and this feature had a marked effect in determining the amount of damage.

The Palace Hotel, an 8-story structure, occupying a large lot, was of this type, honeycombed with interior partitions of solid brick walls, tied together with numerous iron anchors. This building withstood the shock well, but a similarly constructed building at Agnews' Asylum was completely wrecked. Buildings with hollow-block walls were few in number; one of 4 stories in Oakland, one at San José, and four of 2 stories at Palo Alto, are all that have been noted, although others may exist.

Type 3.—Masonry Walls, Metal Interior Frames, and Fire-Proof Floors.—A list of the buildings of this type in San Francisco, with details of construction, is given in Table 1.

Outside of San Francisco, this type was not used, except for some County Court Houses. The older fire-proof buildings were of this type. Generally, the walls were self-supporting only, columns carrying all floor loads. This applies to buildings of 8, 9, and 10 stories. In buildings of less height, generally, the floor load next to the wall was carried by the wall. Data of the construction are given in Table 1. Very little attempt at bracing was used in this type, the Crocker Building offering the only instance of systematic design, and that not being very effective. All had fire-proof floors and partitions.

Type 4.—Structures with a Steel Frame Supporting all Wall and Floor Loads.—A list of these structures in San Francisco, with

TABLE 1.—BUILDINGS OF TYPE 3 IN SAN FRANCISCO.

Name of building.	Stories.	Approximate area.	Use.	Front Walls.		Columns.	Floors.	Ceilings.	Column protection.	Partitions.
				Lower.	Upper.					
Academy of Sciences.	6	7 500	Museum Offices.	G.	S. S.	Cast-Iron.	Concrete.	None.	Metal Lath.	Metal Latb.
California Casket....	7	10 700	Under constr'n.	S. S.	S. S.	Channels.	"	"	Concrete.	None.
City Hall.....	.....	.....	.....	B.	B.	.....	Misc.	Misc.	.....	.....
Crocker	11	10 000	Offices.	G.	B. & T. C.	Phœnix.	Tile.	Tile.	T. C.	T. C.
Crocker Estate.....	5	.....	Lofts.	T. C.	B. & T. C.	Cast-Iron.	Concrete.	None.	None.	None.
Emporium.....	6	.....	Store. Offices.	S. S.	S. S.	Z-Bars.	Tile.	Tile.	T. C.	T. C. and Plaster Bl.
Ferry Building..	3	112 000	.....	S. S.	S. S.	Cast-Iron.	Concrete.	None.	None.	B.
Hall of Justice.....	5	.....	Offices.	S. S.	B.	Pl. and Angles.	"	Metal Lath.	Metal Lath.	Metal Lath.
Hibernia Bank	2	17 000	.....	G.	G.	.....	"	" "	.....	.....
Kamm.....	7	9 000	Lofts.	S. S.	S. S.	Channels.	"	None.	Metal Lath.	Metal Lath.
Majestic Theatre.....	.....	.....	.....	B.	B.	.....	Tile.	Metal Lath.	.....	.....
Mercantile Trust.....	3	5 500	.....	G.	G.	.....	Concrete.	" "	.....	.....
Mills.....	11	21 000	Offices.	M.	B. & T. C.	Z-Bars.	Tile.	Tile.	T. C.	T. C.
Mutual Life	9	5 000	"	G.	B. & T. C.	"	"	"	T. C.	T. C.
Old Chronicle	9	.....	"	S. S.	B.	Cast Iron.	"	"	T. C.	T. C.
Post Office.....	3	68 000	.....	G.	G.	Channels.	Concrete.	Metal Lath.	.....	.....
Scott.....	6	20 000	Lofts.	S. S.	S. S.	Cast-Iron.	"	None.	None.	None.
Sloan	6	.....	.....	T. C.	B. & T. C.	"	"	"	Metal Lath.	"
Spring Valley	6	19 000	Offices.	G.	B. & T. C.	Z-Bars.	Tile.	Tile.	T. C.	T. C.
Union Trust.....	9	8 400	"	G.	B. & T. C.	Channel.	"	"	T. C.	T. C.
Wells Fargo.....	6	19 000	"	G.	B.	Cast-Iron.	Concrete.	Metal Lath.	Double M. L.	Metal Lath.
Young.....	6	3 000	Lofts.	G.	B.	Z-Bars.	"	None.	Metal Lath.	None.

Abbreviations :—S. S. = Sandstone; G. = Granite; T. C. = Terra Cotta; B. = Brick; M. = Marble.

FIG. 1.—EARTHQUAKE DAMAGE TO LIBRARY OF STANFORD UNIVERSITY.

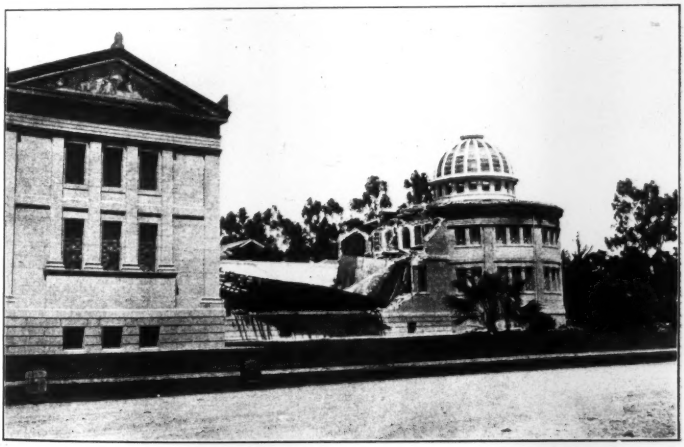


FIG. 2.—EARTHQUAKE DAMAGE TO MUSEUM OF STANFORD UNIVERSITY. REINFORCED CONCRETE SECTION ON THE LEFT, WITH CONNECTING PORTION OF BRICK IN THE CENTER AND ON THE RIGHT

TABLE 2.—Buildings of Type 4 in San Francisco.

Name of building.	Stories.	Approximate area.	Use.	Front Walls.		Columns.	Floors.	Ceilings.	Column protection.	Partitions.
				Lower.	Upper.					
Alexander Hotel	11	4 700	Hotel.	S. S.	S. S.	Channels.	Concrete.	Metal Lath.	Metal Lath.	Metal Lath.
Alto	8	1 500	Offices.	B.	B.	"	"	" "	" "	" "
Aronson	10	9 350	Lofts.	S. S.	B. & T. C.	"	"	None.	T. C.	T. C.
Atlas	10	2 700	Offices.	B.	B.	"	"	Metal Lath.	Metal Lath.	Metal Lath.
Bullock and Jones	8	4 000	Lofts.	T. C.	T. C	"	"	None.	T. C.	" "
Claus Spreckels	19	5 600	Offices.	S. S.	S. S.	Z-Bars.	"	Metal Lath.	T. C.	T. C.
Fairmount Hotel	6	66 000	Under constr'n.	G.	T. C.	Channel.	"	" "	Metal Lath.	Metal Lath.
Grant	8	5 500	Offices.	T. C.	T. C.	"	"	" "	T. C.	T. C.
Hamilton Hotel	12	3 100	Hotel.	S. S.	S. S.	"	"	" "	Metal Lath.	Metal Lath.
James Flood	11	40 000	Offices.	S. S.	S. S.	Z-Bars.	Tile.	" "	B. & T. C.	T. C.
Kohl	11	9 000	"	S. S.	S. S.	Channels.	Concrete.	" "	Metal Lath.	Metal Lath.
Marston	8	1 200	Under constr'n.	B. & T. C.	B. & T. C.	"	"	.....	Concrete.	.....
Merchants Exchange.	14	25 000	Offices.	G.	B.	Pl. and Angles.	"	Metal Lath.	Metal Lath.	Metal Lath.
Monadnock	10	18 000	"	B.	B.	Channels.	"	" "	" "	" "
Mutual Savings Bank.	12	5 000	"	S. S	S. S.	Z-Bars.	"	" "	" "	" "
New Chronicle	16	5 600	Under constr'n.	B.	B.	Pl. and Angles.	Tile.	Tile.	T. C.	T. C.
Rialto	9	16 000	Offices.	T. C.	B. & T. C.	Channels.	Concrete.	Metal Lath.	Metal Lath.	T. C.
Ruef	8	1 000	Under constr'n.	.....	.....	"	"	.....	.....	.....
Shreve	11	9 000	Offices.	S. S.	S. S.	Z-Bars.	"	Metal Lath.	Concrete and T. C.	T. C.
St. Francis Hotel	14	22 000	Hotel.	S. S.	S. S.	"	"	" "	Concrete and T. C.	T. C.
Telephone Co	8	2 500	Offices.	B.	B.	Channels.	"	" "	Metal Lath.	Metal Lath.

Abbreviations:—S.S. = Sandstone; G. = Granite; T. C. = Terra Cotta; B. = Brick; M. = Marble.

details of construction, is given in Table 2. Outside of San Francisco, there were but few: one in Berkeley, of 7 stories; two in Oakland, the highest having 11 stories; one in San José under construction (steel frame up); and the Spreckels Sugar Refinery near Salinas, of 5 stories. All were built with fire-proof floors and brick walls, with brick, stone, or terra cotta fronts. The frame was of steel in all cases noted.

Type 5.—Reinforced Concrete Structures.—Only a few structures of this type existed in the territory affected, and these were built a number of years ago. The old portion of the Museum, and Roble Hall, at Stanford University, were built after the Ransome system; but little reinforcement was used in the walls. In Alameda there was a 4-story factory built on the same system. At Redwood City there was a small 2-story sub-station, and in San Francisco a building was in course of construction with reinforced concrete columns and floors, but with brick walls. The Academy of Sciences had a complete reinforced concrete floor system on cast-iron columns. There was a 1-story factory in Berkeley, and a 3-story factory near Vallejo; also, two residences in country towns. As far as known, these nine instances are all there were in the earthquake region.

II.—The Earthquake Damage.

It is probable that the subject of earthquake damage will be of less interest than that of the fire damage. While considerable that has been learned can only be applied in earthquake centers, still there are many interesting points applicable to other regions. Again, while it is admitted that the Pacific Coast of the American Continents is subject to earthquakes, they are by no means limited to that region. The Charleston earthquake or the one early in the last century in the Mississippi Valley, while not as severe as that in California, might have been sufficient to destroy water supplies and start fires in any large city, the effect of which might have been as destructive as at San Francisco or as at Valparaiso.

Authorities are agreed that ground vibrated by an earthquake has both a vertical and a horizontal motion. The amount has been stated as 1 in. and 3 in., respectively, in the shock under consideration. The amount of motion undoubtedly varies, depending upon the character of the ground. Filled lands and alluvial plains undoubtedly have more motion than rocky hills. By these two motions and their combinations, a building may be damaged in two ways. If large in plan, one portion may be lifted and dropped before the remaining part moves, thus distorting the frame. In either case, a swaying motion is set up, the effect of which is more or less destructive, depending upon the elasticity of the structure, the same stresses occurring as when a live load passes over a truss.

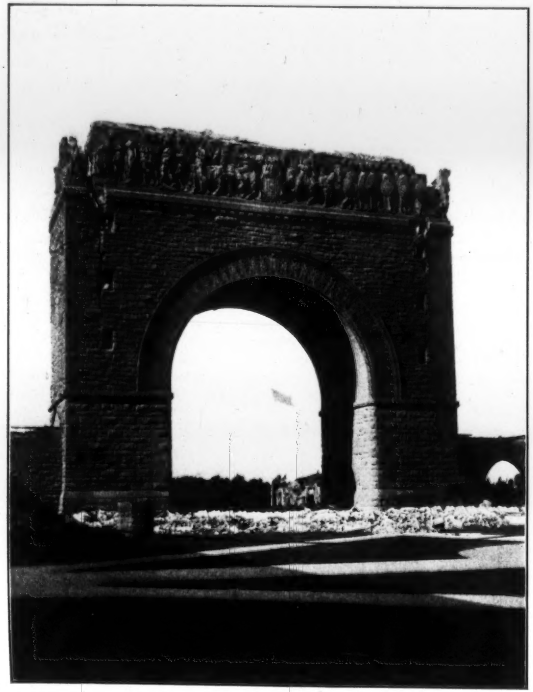


FIG. 1.—EARTHQUAKE DAMAGE TO MEMORIAL ARCH, STANFORD UNIVERSITY.

FIG. 2.—EARTHQUAKE DAMAGE TO BRICK ARCHES AND PARTITIONS, STANFORD UNIVERSITY.

The damage to buildings by the earthquake can now be taken up by considering each type as designated in Part I.

Type 1.—Wood Frame Structures.—The principal damage to buildings of this type was the cracking of plaster and the wrecking of brick chimneys, which occurred everywhere. Numbers of houses were distorted and damaged, by the movement of filled ground upon which they stood, enough to compel their being taken down. This ground vibrated much more than other kinds, and resulted in upsetting houses. Numbers of houses with rotten foundations were upset, and in some cases 3-story structures lurched forward, breaking at the second-floor level and collapsing the first story.

Speaking generally, there was little damage to wooden frame buildings. Undoubtedly, they should be well braced, and the foundation should be carried below the soil to a firm material. Mention might be made of the precaution that pictures and other ornaments in sleeping apartments should not be placed so that in falling they may injure the sleepers. The greatest source of danger lies in brick chimneys. Innumerable instances occurred where the chimney broke through the roof. It may seem paradoxical to say that if brick chimneys must be used, they should be built with poor mortar above the roof line, but the reason is that, if so built, they will be shattered and will fall in parts. If well built, and falling as a whole, the effect is much more destructive. It is idle to expect the exposed portion of a chimney of ordinary dimensions, even of brick and Portland cement, to stand, when vibrating with a house during an earthquake. Some statements are added in the general conclusions following.

Type 2.—Masonry Walls and Timber Interior.—Buildings of this type suffered most from the shock, and in them the greatest loss of life occurred. The damage ranged from minor cracks in brickwork to the complete destruction of large buildings. The swaying motion produced by the earthquake caused diagonal cracking in all directions. In all structures with brick, coursed-stone, or concrete-block walls, these diagonal cracks appeared, crossing each other and generally running at about 45° with the horizontal. Piers between windows, spandrels between window heads and sills, and interior partitions were thus affected. Stone mullions, sills, and transoms were cracked, and large sandstone and granite blocks were split.

Another feature was the shearing of brick walls near the ground. Walls of any length would be sheared from a just-to-be-seen crack up to an amount that upset the entire wall by removing the support.

Gables were upset everywhere. As built, there was little if any connection between the wooden roof and the brick. As the

wall was thin, it resulted in upsetting the gable. The most marked instance of this was the gable wall of the Memorial Church at Stanford University, which stood, practically disconnected from the wooden roof. It was ornamented with a costly mosaic, and the entire wall fell.

Buildings of this type also suffered by the wrecking of fire-walls and cornices, which, falling in or out, did much damage. Some fire-walls, falling on lower buildings, wrecked them. People were killed in the street by cornices and upper walls falling outward.

A favorite method of constructing such buildings, when used as stores, was to build the front wall upon a girder, placed at the second-floor level, and supported on columns. As the interior frame was generally independent of this girder and the wall, the result was that the wall fell outward.

Another cause of the destruction or partial wrecking of such buildings was the roof construction. Cases were observed where roofs, of as much as 80 ft. span, were built with inclined rafters resting upon the walls and unprovided with horizontal ties of any kind. When the building was in motion, the inclined rafters acted as thrust pieces to overturn the walls.

The foundations of such buildings were generally found intact, the damage increasing toward the top. Many arches were opened at the crown, and the arch stones or bricks dropped.

It must be recognized that buildings of this type were general throughout the region of the shock. The damage ranged from slight cracks to the complete destruction of large structures, hence no general statement covers the case. A few individual statements may be of interest. In Oakland, a concrete-block building, 4 stories high, was injured little if any. At Palo Alto, two such structures, 2 stories high, probably not very well built, were completely wrecked, the blocks being broken and disintegrated.

At the Agnews' State Asylum, large 4-story structures were completely wrecked. The walls stand, but are shattered everywhere. At this place, all partition and hall walls were brick, the buildings being a series of cells. It is difficult to describe the destruction of this brickwork. Probably 90% of the mortar bonds were broken, leaving the wall but a pile of loose bricks. The mortar was of fair grade. A large tower of brick over one building fell, doing much damage. At San José, the large City Hall, of brick, with brick partition and hall walls, was shattered everywhere. The old County Court House was not badly hurt, but two later buildings were badly shattered.

At Stanford University, the uniform method of construction was brick walls, with a veneer of coursed ashlar. The buildings

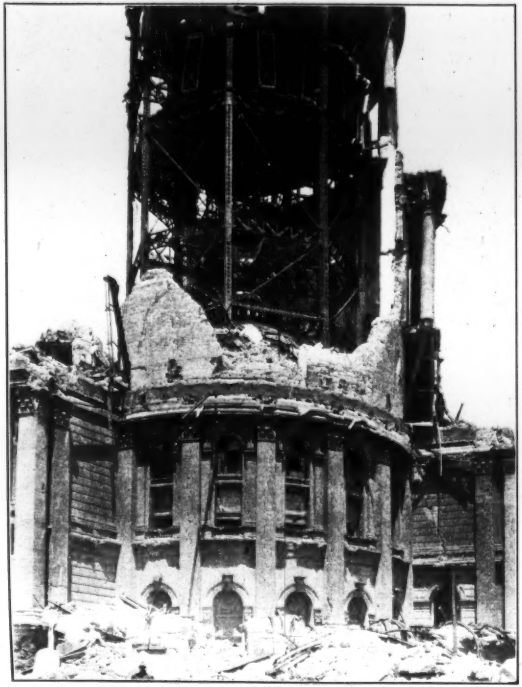


FIG. 1.—EARTHQUAKE DAMAGE TO CITY HALL, SAN FRANCISCO. NOTE THAT STEEL FRAME DID NOT SUPPORT THE WALLS.

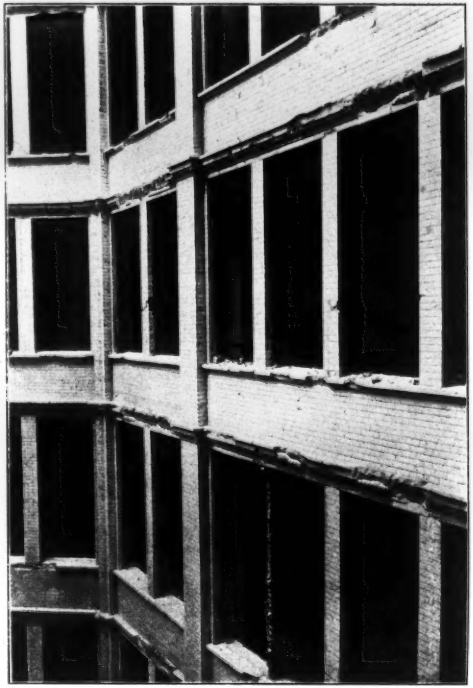


FIG. 2.—BREAKING OF TERRA COTTA BELT COURSES AND SILLS IN JAMES FLOOD BUILDING.

were connected by numerous arcades formed with two rows of arches covered by a hip roof. The arches rested upon single stone columns. Several buildings here were completely destroyed, notably the large Library, Gymnasium and Museum buildings. The arches were injured by the dropping of keystones, the movement of supporting columns and the falling of spandrels. In many of the walls there was a separation of the ashlar facing from the brick backing. The cracking of walls, with other damage as noted above, was evident everywhere. Foundations were nowhere injured.

At St. Patrick's Academy, at Menlo Park, some large 4-story brick buildings were damaged, the worst effect being in the two upper stories. Two-story structures were but little injured. A tall tower on one building was badly wrecked.

At Santa Rosa, nearly every brick building was wrecked, and the resulting fire destroyed the entire business part of the town.

In San Francisco, while there are some special instances of buildings of this class withstanding the shock, there are others where there was complete destruction. The Palace Hotel is a notable instance of how a brick building may withstand the earthquake, it being practically uninjured.

Type 3.—Masonry Bearing Walls, Metal Interior Frames and Fire-Proof Floors and Partitions.—It may be questioned why this type was separated from Type 4, with a steel frame carrying all loads. The reason is that buildings of this type were damaged more by earthquake and fire than those of Type 4. In Table 1, showing the buildings of this type, no distinction is made between walls carrying some of the floor loads and those where all floor loads were carried on a steel frame—columns being placed in the walls. Of the latter class, the Mills, Crocker, Kamm, Union Trust and Mutual Life Buildings are examples. The frame of the Mills Building (1891) has columns of one-story section, each section resting upon the cap of the one below, and connected thereto with four bolts. Beams and girders rest upon the cap plate, and the entire structure was bolted above the third floor. After the earthquake and fire, this building leaned from 7 to 9 in. out into Bush Street. The distortion of the frame was uniform above the second floor. There being no crack in the outer walls, it is difficult to say whether this movement was due to the shock, or had been a gradual movement from the time of construction. As all buildings of this type were burned, it is difficult to separate earthquake from fire damage.

In the Kamm Building, one wall, 7 stories high—60 ft. of 13-in. wall and the remainder 17-in.—sprung out about 14 in. near the middle height. The reason is obscure, some stating that it was caused by the explosion of dynamite. In other respects, the outer

walls of buildings of this type suffered as other brick walls. Interior partitions of terra cotta show all the characteristic cracks of brick walls, the mortar joints being broken and opened from ¼ in. to 2 in. Cornices, being generally supported on iron brackets, held, but here, also, it is difficult to separate fire from earthquake damage. Speaking generally, those buildings with a steel wall column supporting the floors suffered less than those where the walls held the floors.

Type 4.—Structures with a Steel Frame Supporting all Wall and Floor Loads.—Relatively, buildings of this type were those most exposed to earthquake damage, the type including most of the tall buildings in the city. Here, as elsewhere, it was a question of bracing. Some frames were well braced, others partially so, others not at all. The Claus Spreckels Building (19 stories), about 80 ft. square in plan, was braced better than any. Portal bracing was placed on each side of each corner, up to the third floor. In addition, there were eight lines of eye-bar diagonals from the basement to the fourteenth floor and three above. All girders had gusset-plate knee-braces. The Mutual Savings Bank Building (12 stories) had two lines of portals and two lines of eye-bar diagonals from top to bottom. The New Chronicle (16 stories) had deep girders and gusset-plate knee-braces throughout. The Shreve Building (11 stories) had deep girders and two complete systems of portal bracing. The Kohl Building had deep lattice girders throughout. The Merchants' Exchange, St. Francis Hotel, Fairmount Hotel, Monadnock, Rialto, and James Flood Buildings were large in area, and were without special wind bracing.

Earthquake damage consisted in cracks in partitions, where made of tile. On the buildings where corner piers of stone rested upon concrete under the sidewalk, the stonework was badly cracked. Where such stone was carried on the steel frame, no cracks are found, but there are only a few instances of this method. Walls of brick, or of stone and brick, did not suffer as much as might have been expected. The rear walls of the St. Francis Hotel were cracked. The walls of the Atlas Building were affected in the same manner. The rear brick walls of the Merchants' Exchange moved outward on the girders from 1 to 3 in. On the Spreckels Sugar Refinery, near Salinas, there were several cases of the 13-in. brick walls being thrown down, but no such damage occurred in San Francisco. A number of piers and spandrels of the brick front of the new Chronicle Building were cracked, as were also the terra cotta faces of the Fairmount. The brick front of the Monadnock was badly cracked. On the Claus Spreckels Building, the sandstone fronts were intact up to the tenth story. From there to the sixteenth story, the stonework was badly worked on the beds.

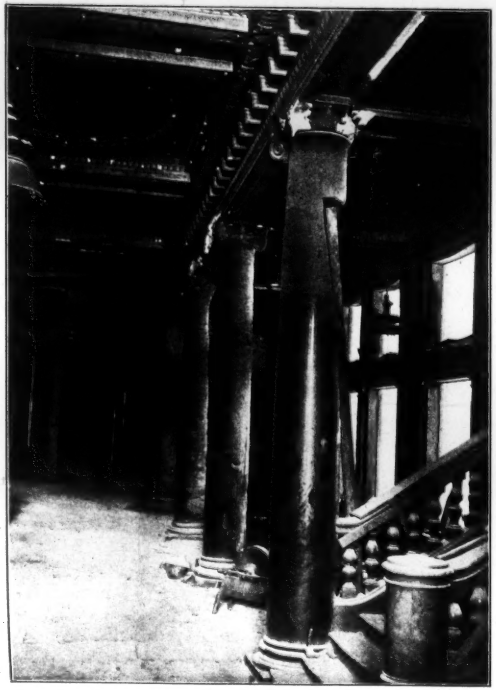


FIG. 1.—FIRE DAMAGE TO INTERIOR GRANITE COLUMNS IN JAMES FLOOD BUILDING.

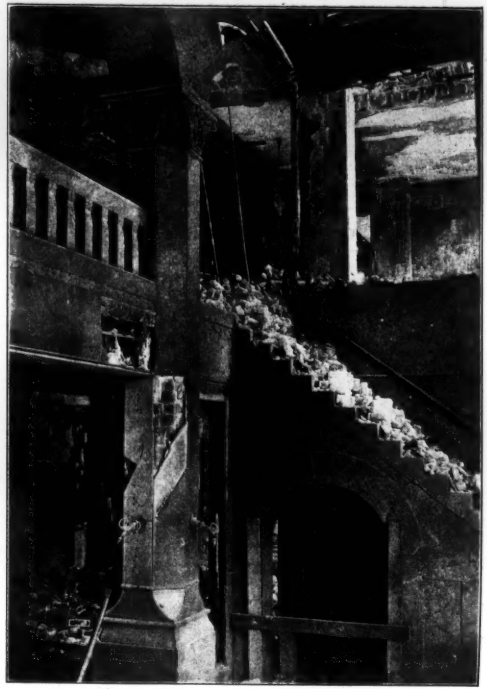


FIG. 2.—FIRE DAMAGE TO MARBLE STAIRWAY IN MILLS BUILDING.

Some stones were split, and nearly every one moved, the maximum movement being possibly 2 in. This stonework rested on girders, and was backed with brick. The dome of the building containing the seventeenth, eighteenth and nineteenth floors, was covered with terra cotta, each piece being anchored to the metal frame. As far as could be determined, no damage or movement of this terra cotta took place. The same is true of the terra cotta gables of the Mutual Savings Bank. As far as observed, the damage to stone facing was greatest on the Claus Spreckels Building.

To the statement of damage to stonework, exception must be taken in the case of stone cornices. Elaborate cornices of sandstone were used on the Claus Spreckels, Mutual Savings Bank, Kohl and James Flood Buildings, all supported on iron brackets, and no movement or breaking of stone could be observed. The same is true of terra cotta cornices on other buildings; in fact, the care taken to support cornices seemed to protect them from damage.

No foundation damage was observed in any buildings.

The damage to steel frames was almost negligible. Some rivets were sheared in the James Flood Building, and in the Union Trust Building a number of bolts were sheared. As most of the frames are hidden, the extent of the damage is unknown, but is probably small. A few of the eye-bars of the diagonal bracing of the Claus Spreckels Building were bent, as if the opposite ones had taken the strain. A careful examination failed to reveal any sheared rivets. The greatest damage to a steel frame was that of the tower of the Ferry Depot. This tower was about 40 ft. square and some 200 ft. high above the main structure. It was sheathed with brick and stone, except the top, which was of metal. The frame had a column at each corner, with a complete system of diagonal rods. A number of rods were broken, and in places the gusset-plates were torn and the rivets sheared. This building rests upon piles driven into deep mud, and it was probably badly shaken.

Type 5.—Reinforced Concrete Structures.—Only a few instances of this type exist. The old portion of the Stanford University Museum, a complete structure, was uninjured, except for a few cracks. The connecting brick portion was badly shattered. Roble Hall, at the same place, a 3-story and attic structure, was uninjured, except where some brick chimneys fell, breaking the roofs and floors. The sub-station at Redwood City was injured, and a part had to be rebuilt. This structure was built on marshy ground near the bay, and was exposed to the worst effects of the shock. A warehouse in San Francisco, in course of construction, with two stories up, had its brick walls wrecked, but the reinforced concrete frame and floors were uninjured, except by the fall of the

walls. The floors of the Academy of Sciences and of the Folger Building were uninjured by the shock, and the same is true of the complete reinforced concrete building, 3 stories high, in Alameda. The factory buildings and residences before mentioned were uninjured. These constitute the only examples of this type known to the writers, but all the evidence points to the superiority of this method of construction.

Conclusions.—The writers offer the following as conclusions based upon a study of the effects of the earthquake. These conclusions are offered for the reason that it is recognized to be impossible to set forth in a partial description a full idea of the damage.

The effect of the earth motion is to set a building in motion. The structure is thus subjected to all the stresses occurring in a truss sustaining a live load. The amounts of the stresses are unknown, and cannot be predicted, as the intensity of the shock is unknown. Obviously, the shock may range from a tremor to that of a violence that would wreck any building. Again, should the earth-slip take place beneath a building, it would be wrecked. Sufficient evidence is at hand to warrant the statement that a building designed with a proper system of bracing to withstand wind at a pressure of 30 lb. per sq. ft. will resist safely the stresses caused by a shock of an intensity equal to that of the recent earthquake in California.

The prime requisite of the structure is elasticity. This must be understood as the ability of a structure to return to its original form after distortion. This elasticity allows the building to absorb the motion of the earth, where a more rigid structure would be ruptured.

To this requirement, the building with a timber or steel frame answers very well. The reinforced concrete structure does so also, with the exceptions noted below. The building with stone, brick or block construction, having horizontal mortar joints, does not answer the requirement at all. It may be stated, as one of the most obvious lessons of the earthquake, that brick walls, or walls of brick faced with stone, when without an interior frame of steel, are hopelessly inadequate. As a method of building in earthquake countries, such types are completely discredited.

To resist the shearing effect of the horizontal earth motion, vertical members are necessary. The shear is transformed into diagonal forces, which appear as stresses in diagonal and horizontal members. There was probably no better illustrated lesson of the existence of diagonal stresses than that offered by the innumerable instances of the cracking of brick and stonework along diagonal lines. In relation to this, it may be stated that a brick spandrel wall adds little, if any, to the bracing of a steel frame.

PLATE XXX.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LIX, No. 1056.
SAN FRANCISCO EARTHQUAKE.
REPORT ON BUILDINGS.

FIG. 1.—SPALLING OF MARBLE FRONT BY FIRE.

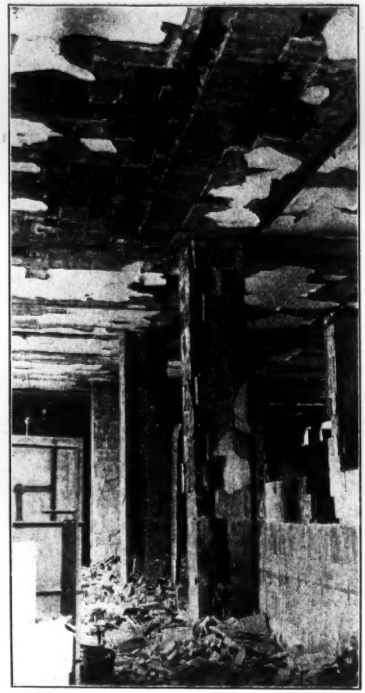


FIG. 2.—FIRE DAMAGE TO TERRA COTTA TILE PARTITIONS AND COLUMN FIRE-PROOFING.

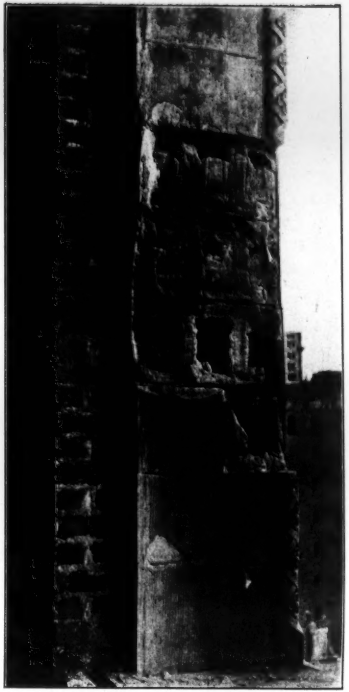


FIG. 3.—FIRE DAMAGE TO TERRA COTTA WINDOW TRIM. A GOOD EXAMPLE OF AVERAGE DAMAGE TO TERRA COTTA AROUND WINDOWS.

Many of such walls were cracked badly, and moved on the supporting girder. No reliance should be placed upon them, as they are open to all the objections stated in connection with brick walls in general. The well-designed steel frame offers the best solution of the question of an earthquake-proof building, as all the stresses can be cared for. The well-designed timber-framed house is also adequate. A reinforced concrete building offers a solution, but is open to the following objections:

Architectural reasons demand that diagonal bracing shall not be used except on rare occasions. This is overcome by the use of gusset-plate knee-braces and portal-braces in the steel frame. Such design induces severe bending moments in the columns and girders; and, in the girders, the moment may be of the opposite character to that of the floor loads, thus producing tension in the upper flange of the girder. As at present designed, no reinforcement is used at that point, and hence such a girder would be defective. Again, it will be found upon analysis that relatively great stresses occur at points where the girders join the columns, especially in the lower floors of tall buildings. Here, again, the reinforced concrete construction, as now designed, is weak. These remarks are offered more in connection with high buildings. They can be overcome by the designer in reinforced concrete. All the evidence in the recent shock favors reinforced concrete, but the writers are of the opinion that the steel frame offers the best solution of the problem.

Foundations did not suffer at all, no instances of any damage having come to hand. Some discussion has taken place as to the advisability of making a monolithic mass under buildings. Several of these have been constructed, such as the Claus Spreckels, Mutual Savings Bank, and Bullock and Jones Buildings. They are all of relatively small base. It is commercially impossible to construct a monolithic base under a building, say, 12 stories high, and having a base of 150 ft. Buildings of that size and larger, with isolated pier foundations, suffered no more than others. The evidence is that foundations well built, along accepted lines, are adequate. It might be claimed that, if such had been used in large structures, the damage would have been less. The evidence does not point that way, for even if the monolithic base were sufficiently strong to resist the vertical earth motion, the horizontal motion would still vibrate the structure.

Evidence for floors is not conclusive, as all terra cotta arch floors were afterward burned. Terra cotta arches covered with concrete stood without much damage in the brick portion of the Stanford University Museum. The terra cotta there could not be seen. Analogy with masonry walls would seem to say that many

of the joints would be broken. Ordinary concrete floors stood the shock with but little damage.

In the case of partitions, those of terra cotta tiling were everywhere cracked and opened. It amounted to practical destruction in most cases. In this case, earthquake damage can be distinguished from fire damage. Partitions of metal studs and lath suffered less, but plaster was badly cracked. Nothing seems to be suggested for a partition, in which the plaster would not be destroyed.

For rear walls, reinforced concrete offers the best solution, the reinforcing members being tied to the steel frame. A facing of brick or stone could be backed with reinforced concrete. In the case of stone, the parts should be doweled together, and, if possible, all tied to the steel frame. Terra cotta as a facing for walls is admirable, in this respect, as it offers superior facilities for tying it to the steel frame. For fire and parapet walls, the steel frame should be carried up, and anchors should be provided.

Brick chimneys, large and small, are open to all the objections of brick walls, only in a more marked degree, owing to their isolated design. Reinforced concrete seems to offer the best method for such construction.

Arches with voussoirs are not able to resist earthquakes. The motion opens the joints, and the keystones fall, thus thrusting aside the abutments. Evidences of this exist everywhere.

Finally, it may be questioned whether difference in workmanship was not responsible for many of the results. While it is true that good workmanship gave better results than ordinary, it is still the opinion of the writers that it was mainly a question of design. Agnews' Asylum was of brick, laid in a fair grade of lime mortar. Ten miles away, on similar ground, St. Patrick's Academy, of similar design, was of brick, laid in lime and Portland cement, and there was better work than at Agnews'. The damage at the latter place was less than at the former, but, as far as use was concerned, both places were demolished. The tower at St. Patrick's Academy was of brick, laid entirely in Portland cement mortar, and the work was so well done that the brickwork invariably broke through the bricks and not at the joints; yet the tower was completely destroyed; in fact, it was the worst wrecked of all the buildings there.

The writers simply reiterate the statement that, speaking generally, buildings of brick walls and wooden interiors cannot be built which will not be wrecked in a severe shock, it being a fault of design and not of materials or workmanship.

III.—The Fire Damage.

Within half an hour after the earthquake shock, the Fire Department of San Francisco had notice of fifty-two different fires.

FIG. 1.—FIRE DAMAGE TO TERRA COTTA ORNAMENT. NOTE GOOD CONDITION OF MOULDED BRICK.

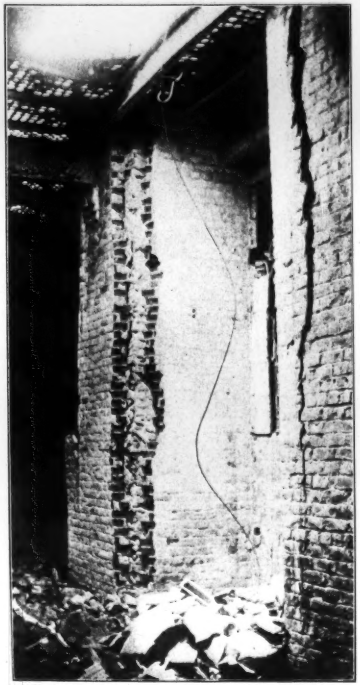


FIG. 2.—SPALLING OF CORNER BRICK PIERS IN BASEMENT OF MILLS BUILDING.

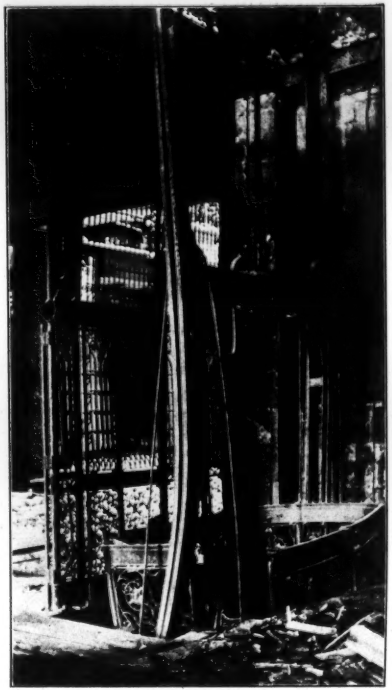


FIG. 3.—FIRE DAMAGE TO ELEVATOR APPLIANCES IN MILLS BUILDING.

It is probable that others had started. The fire lasted three days, being finally stopped by dynamite, wide streets, and individual efforts. The wharves of the city were saved by State and United States Navy tugs. Only a portion of the residence district was saved. Some isolated fires were extinguished by the Fire Department, using water in the dead ends of mains. The destruction of the supply mains, described in another report, was complete, and hence the entire fire burned without interference. The area destroyed was a trifle in excess of 4 sq. miles. It was entirely covered with buildings, excepting where there were lumber yards. A few isolated blocks escaped, but they are of no interest in connection with this report.

In commenting upon the destruction by this fire, it must be borne in mind that no water was used on any of the materials. Hence, in some cases, the damage is less than it might have been had water reached the materials while they were hot.

Of the buildings within the burned area, estimated at nearly 20 000, only those given in Tables 1 and 2—some 50 or 60—remain standing. All others are completely destroyed, only some brick walls standing here and there. Unless one has been an eye-witness, it is difficult to realize how all materials that men make into the shape of buildings can be so utterly destroyed in a general conflagration. With two exceptions—the Kohl and Atlas Buildings—all the buildings in Tables 1 and 2 were completely burned in the interior, only the frame remaining.

All the business center of the City of Santa Rosa was destroyed, but as only buildings of Type 2 were burned, nothing additional can be learned there, and no further mention will be made of this place.

It would be well to state here that when the phrase "complete destruction" is used, it means that the part is destroyed so that it is unfit for further use. Under this, a disintegrated terra cotta floor, or a leaning brick wall, or a bent steel beam is destroyed, even if the part may be again reshaped and used.

No attempt will be made here to describe in detail the amount of destruction of any one building. Cases will be cited illustrating various points. It is believed that a description of the fire effects on different materials and upon their combination will be of more real interest. Reference will only be made to buildings of Types 3 and 4. Types 1 and 2 were completely destroyed, and Type 5 had no example of moment. In Type 2, the city ordinance allowed buildings 100 ft. high, if lathed with metal; and of 82 ft., if with wooden lath. No evidence is offered regarding this, as they were all destroyed.

Bearing in mind the definition of "complete destruction," it

can be said that all materials were destroyed when directly exposed to the fire for any length of time. Brick, terra cotta, granite, marble, sandstone, steel, cast iron, concrete, plaster, cement, timber —all failed. Especial emphasis is placed on this point. Anyone with an interest in any one material can find evidence to support a claim for it; yet, impartial observation leads to the conclusion that, when a material was subjected to what would be considered a test fire, it was destroyed.

Front Walls.—All ordinary methods of construction were exposed to the fire. Granite suffered more than any other stone, spalling badly wherever exposed. Marble fared nearly as badly, and sandstone slightly less. With all, there was complete destruction. No limestone fronts had been used. Of the terra cotta fronts, most were destroyed, for instance, the Bullock and Jones Building. Glazed terra cotta blocks, used in court walls, were generally cracked. Terra cotta brick spalled everywhere. The walls of the Mills and Merchants' Exchange Buildings are to be removed. Moulded terra cotta brick and pressed brick withstood the fire best of all, but were spalled where much exposed. Terra cotta brick with rounded corners fared best, sharp edges cracking off.

Either stone, brick or terra cotta was used around windows, and here the damage was the worst. In relation to this, it may be said that terra cotta is deceptive, in that it retains its form after being destroyed. Many fronts, apparently in good order, must be removed. In the Mills Building, there was hardly a window opening in which the terra cotta sills, jambs and heads were not badly cracked. From the street, they had the appearance of being in good order. Walls of ordinary red brick and mortar were destroyed in many cases, especially in buildings of Type 2, where the fall of the interior frame pulled down the wall. There are many instances of the integrity of such walls, especially at corners; while many of them were injured in the earthquake, it is not believed that this was a factor of much importance in the fire. The damage to standing walls in all buildings was the spalling of the red brick, sometimes to a depth of 1 in.

Foundations.—No foundations were found injured by the fire.

Floors.—Only two kinds of floors were exposed: terra cotta and concrete. Of the two, terra cotta suffered the more, being in all cases, where directly exposed, completely destroyed. Terra cotta floors, built with flat arches and with plaster applied directly, were all destroyed. The general effect was a spalling of the lower section of the tile structure and a disintegration of the mortar joints. The tile also became brittle, breaking with a slight blow. All terra cotta was of the dense type, no porous tile being used. In the James Flood Building, tile arches were used, being of the regular

FIG. 1.—BUCKLED COLUMN IN FAIRMOUNT HOTEL.

FIG. 2.—BUCKLED COLUMN IN BASEMENT OF ARONSON BUILDING.

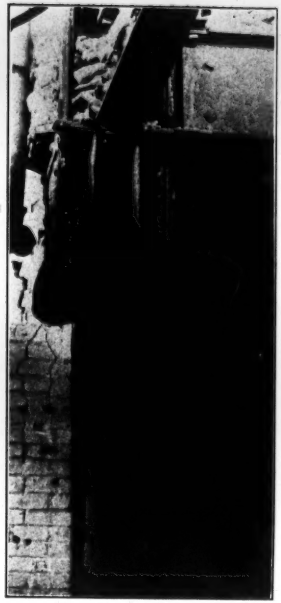


FIG. 3.—BUCKLED COLUMN IN MILLS BUILDING. TERRA COTTA TILE PROTECTION.

FIG. 4.—FIRE DAMAGE TO PHŒNIX COLUMN IN CROCKER BUILDING.

arch construction. They were covered with 4 in. of concrete, and protected by a wire-lath ceiling. They escaped injury except where the ceiling lath fell, and there the lower layer spalled off.

Concrete floors generally had hung ceilings, and, where thus protected, were uninjured. Where exposed, the concrete is in most cases destroyed, for instance, in the Sloan, Rialto, and Aronson Buildings, and the Crocker Warehouse. The concrete is dry, and while in many cases hard, yet all the water has been burned out and it may be said to be destroyed, even if able to support weights. Floor coverings of wood invariably burned, adding to the destruction. Sleepers were generally burned. Surfaces of cement mortar fared much better, the linoleum covering remaining practically intact.

Partitions.—Partitions were of terra cotta tile, and of wire-lath and plaster, either solid or hollow. All kinds were destroyed. In the tile partitions the mortar joints were disintegrated, the plaster was destroyed, and the tiles were made brittle. One could pull down with the hand any partition in the Mills Building, all of which were of tile. Metal lath and plaster partitions were completely wrecked, but the lath might be considered as salvage. The use of wooden grounds around doors and transoms helped the destruction, but it is difficult to see what would have prevented the damage.

Steel Frame.—The steel frames were the least injured of any part of the various structures. Where properly protected, there was no injury. Where the protection was faulty, or where there was none, the destruction was complete.

Cast-iron columns were used in a few buildings of Type 3. As a rule, not as much care was taken to protect them as in the case of steel columns. Numbers of cases have been observed of the melting of cast-iron columns, especially lugs, in buildings of Type 2. In Type 3, the most conspicuous example is that of the Sloan Building, used as a carpet and furniture store. A number of columns in the basement, covered with one layer of lath and plaster, melted so that the metal ran together into a lump, lowering the floors from 1 to 4 ft., and completely wrecking the building. The basement was full of linoleum. In the Wells Fargo Building, with cast-iron columns protected by double lath and plaster, and used as offices, no damage resulted.

Steel columns fared about the same. In the Fairmount Hotel, probably 25% of the columns was destroyed. These columns had practically no protection, being built in between lath and plaster partitions. These partitions had interior blind spaces of from 6 to 18 in. in width, and, in many of the wall columns, the space extended from the first floor to the roof. To stiffen the partitions,

studding of timber, 2 by 4-in., 4 by 4-in., and 4 by 6-in., was used everywhere. The burning of the timber directly against the unprotected column, in addition to the general fire, destroyed it.

In the Hotel Alexander an apology of wire lath and plaster was used as a fire-proofing. It was cut into everywhere for door frames; and thirty-nine column sections were destroyed. In the Aronson Building, the terra cotta fire-proofing fell away, and ten column sections were buckled. One case of this kind occurred in the Mills Building, and several columns were bent, for the same reason. One column section was similarly wrecked in the Crocker Building, and two sections were bent in the St. Francis Hotel, where the tile fire-proofing had been cut away to allow the placing of rolling doors. It may be stated that every instance of failure can be thus explained.

Steel beams and girders were bent where unprotected, but remained uninjured otherwise. Much of the injury was suffered in first floors; basements and sidewalk beams, as a rule, having the lower flanges unprotected. Comment on protection is offered below.

Most roofs were simply of beam construction, similar to the floors, but the damage was generally less, owing to the protection afforded by hung ceilings. The trusses in the Mutual Life Building were covered with tile, which failed to protect, allowing the trusses to fall and damaging the supporting columns. A number of roofs were destroyed from lack of adequate protection, generally, where made of terra cotta, the Crocker and Spring Valley Water Company Buildings being examples.

Ceilings.—Where ceilings were made by plastering directly upon terra cotta, flat floor arches, they were completely destroyed. Where metal-lath, hung ceilings were used, the plaster was destroyed, but the lath generally remained in place, and was not damaged. Where copper wire was used to fasten the wire lath it failed, dropping the ceiling. Such wire should never be used.

Miscellaneous.—Vaults in offices, where made with doors set in a terra cotta, or wire-lath and plaster box, failed everywhere, the contents being burned. Vaults made with an iron box covered by fire-proofing fared much better. Safe deposit vaults in the basements of banks were uninjured, which suggests similar devices for the tenants of large buildings.

All plaster was destroyed. Marble tiling for floors was a complete failure. Marble treads for stairs failed, as also all wainscoting of that material, the marble being calcined.

Wire-glass covering of skylights resisted the fire well, except where the glass melted. Temperatures that melted glass were reached everywhere.

Metal-covered window frames, doors, and door frames were used

FIG. 1.—CONCRETE BLOCK CONSTRUCTION IN PALO ALTO, DAMAGED BY EARTHQUAKE.

FIG. 2.—FIRE DAMAGE TO INNER COURT OF MILLS BUILDING.

in the Kohl Building, the upper six stories of which were not burned. No opinion is offered, except that this building was not much exposed, the glass of the exterior windows not being even cracked. The metal trim undoubtedly acted to deter the spread of fire within the building.

Steel shutters were used in the Telephone Building, but it was completely burned. It is claimed that the fire entered through some unprotected windows.

Elevator grills were generally destroyed, together with the cages and machinery above the basement.

Some question may arise as to the temperatures during the fire. Glass was melted everywhere. The softening of cast-iron columns in the Sloan Building has been referred to. There were numbers of cases of the actual melting of cast-iron columns, and in many buildings steel was melted. Cases of the latter were observed in a dry-goods store, where numbers of I-beams were fused; and also in a machine-shop, where steel rods were fused.

Conclusions.—Any deductions from the fire must be those based upon a general conflagration, and not those of an isolated fire. In view of the complete destruction of all materials, it becomes a question as to what should be done to make a building fire-proof.

San Francisco was built probably in about the same way as other cities. It is an error to say that it was a wooden-frame city, as the business district was generally composed of buildings with brick walls. In among these had been constructed the so-called fire-proof structures, exposed on all sides to danger by the burning of the inflammable structures around them.

The only statement that can be offered is that the best insurance for buildings would be the isolation of a district containing nothing but fire-proof structures. A general conflagration would then be impossible. Manifestly, this is impossible in San Francisco, where business must be resumed with the least cost. In many cities, it would be good insurance for men owning large buildings to combine to buy out old and inflammable structures, either demolishing or rebuilding them. Otherwise, there remains the danger of general conflagrations, such as those at Baltimore and San Francisco, in which fire-proof buildings will be injured from 30 to 60 per cent.

Turning to the individual building, the question of the exterior walls must be settled. There does not seem to be much choice of material. Architectural considerations demand the use of brick, terra cotta, or stone. With a steel frame supporting the walls at each story, any local fire will destroy the nearby facing, but it may be removed without damage to adjoining parts. This cannot

be done when walls are self-supporting and the facing acts as a part of the wall. This risk of damage must always be carried, gradually becoming less as inflammable buildings are eliminated.

There is no doubt that the steel frame is adequate for all its purposes, but it must be protected. This brings up the general subject of fire-proofing, in which is involved the construction of floors and partitions, and ceiling and column protection. Where any reasonable protection was given the steel frames of buildings in San Francisco, the steel was uninjured, and hence the writers feel warranted in stating that it is possible to protect such a frame so that it will pass uninjured through a fire that consumes all parts that can burn.

It was stated previously that all materials were destroyed by fire; it follows, from this, that the destruction of fire-proofing must be expected, and that it will have to be restored after a fire. It becomes a question, then, of selecting the material that will stand up best, for the fire-proofing must retain its form, even if destroyed.

In the writers' judgment, the column should be of a closed form, such as channels and plates. This is preferable to latticed columns. For columns, the fire-proofing that will stand up best is red brick set in Portland cement mortar. Equal to this is a casing of solid concrete at least 4 in. thick, with a mesh of reinforcing metal. Examples were found in the St. Francis Hotel and Shreve Building, where the concrete was uninjured. In the Fuller Building, which was used as a paint and oil warehouse, the floors were of wood, and the columns were covered with from 4 to 5 in. of concrete. This protection held in place when the columns fell. It was completely destroyed, but the column shafts were protected. Next in order, and of equal merit as far as examples show, is the double wire-lath and plaster protection, which, in the Wells Fargo Building, afforded complete protection. In many buildings, columns were protected by one layer of lath and plaster, directly applied, and then the entire column, with pipes, etc., was enclosed by the regular partition. This afforded complete protection, and the Merchants' Exchange and Kohl Buildings are examples. The examples of the Fairmount and Alexander Hotels are not included, as obvious defects in design and execution warrant the statement that the columns in these buildings were practically unprotected. The same remark also applies to the melted cast-iron columns of the Sloan Building.

The remaining examples of column failures must be laid to the failure of terra cotta tile. The work in the Mills, Crocker, and Aronson Buildings was well done, but in all, and in the latter case especially, it failed utterly to afford complete protection. As the failure of one column section means the practical destruction of all

FIG. 1.—UNITED STATES MINT, SAN FRANCISCO. WITHSTOOD EARTHQUAKE AND FIRE.

FIG. 2.—HALL OF JUSTICE, SAN FRANCISCO, AFTER EARTHQUAKE AND FIRE.

floors supported by that column, the results are serious. In justice to the terra cotta tile, it must be said that in the St. Francis Hotel, the Union Trust, and some other buildings, it stood up well enough to protect the columns. The writers believe, however, that it is the least valuable of all materials commonly used for fire-proofing.

For floor construction, some form of reinforced concrete is far preferable to tile. In all cases the record of concrete is better than that of tile. Connected with this is the protection of the lower flanges of beams and girders. The fire shows that a cover of lath and plaster directly upon the flange, protected again by the suspended ceiling, is the best. The layer of plaster alone on the flange will not protect. Neither will the thin piece of terra cotta strapped on. It may be stated here that one of the most obvious lessons taught by this fire is the protection to concrete floors and floor beams by the suspended ceiling of lath and plaster. In all cases where used, it afforded complete protection. Where not used, concrete was destroyed and beams were distorted.

The subject of partitions is bound up with that of column protection. Terra cotta tiles are inferior to lath and plaster, although both were destroyed. A partition may be destroyed, but, if it stands, it impedes the spread of fire, and, in this light, the lath and plaster type is superior to tile. It should be possible to construct better partitions, but as yet no better ones have been offered.

A logical deduction from the statement that all materials were destroyed is the conclusion that all structural parts of a building, of whatever material constructed, must be protected by another material which will be a more or less complete loss in a fire. This applies to a steel frame, to floors of any type, and to roofs. It is impossible to protect some parts, such as fronts, partitions, and other parts directly exposed. The floors and frame constitute the structural parts, failure of which means destruction of the building. All such should be fire-proofed. This remark applies with equal force to buildings with reinforced concrete columns, girders, beams and floors. As integral structural parts, they should be fire-proofed as well as similar members of a steel-frame structure, for concrete is destroyed by fire nearly as quickly as steel.

No further comment is offered, except the following. Buildings with wooden floors will be completely destroyed in a fire. Such parts as metal trim, wire-glass, and steel shutters, were not used to sufficient extent to warrant any definite conclusions, except to say that what showing there was was favorable. Whether this expense is warranted was not determined. Bad work and indifferent construction will cause any material to fail. Good work will enable a poor material to stand up. Fire-proofing should be continuous, and at no place should it be cut into for the passage of pipes, etc.

The subject of pipes should be treated as it deserves, and proper ducts and shafts should be provided, instead of allowing them to be placed anywhere, where they will give the least trouble.

It is hoped that comment will be received on this report, but it is trusted that it will be in the nature of criticism, and not in the way of advocacy of some one form of construction.*

* This report was accompanied by 122 photographs, from which 34 have been selected for reproduction. They are shown in Plates XXV to XXXIX.

FIG. 1.—FIRE DAMAGE TO RESIDENCES AND HOTELS OF TYPE 2 CONSTRUCTION IN SAN FRANCISCO.

FIG. 2.—DESTRUCTION OF BUILDINGS OF TYPE 2 CONSTRUCTION AT FOOT OF MARKET STREET, SAN FRANCISCO.

FIG. 1.—EXTREME CASE OF EARTHQUAKE DAMAGE TO FRAME BUILDINGS IN SAN FRANCISCO. (IN CENTER.)

FIG. 2.—EARTHQUAKE DAMAGE TO MAIN BUILDING OF AGNEWS' ASYLUM, NEAR SAN JOSÉ.

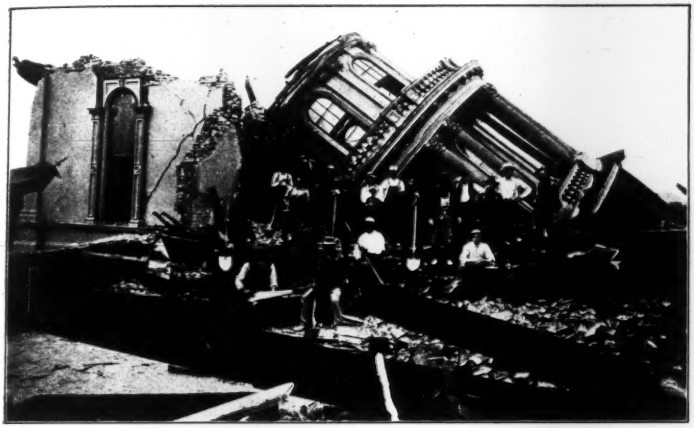


FIG. 1.—EARTHQUAKE DAMAGE, SECOND FLOOR OF COURT HOUSE, SANTA ROSA.

FIG. 2.—EARTHQUAKE DAMAGE IN OAKLAND. NOTE WALL CONSTRUCTION OF TOWER. THIS TOWER WAS NOT DEMOLISHED BY THE EARTHQUAKE, BUT WAS TORN DOWN WITHIN TWO WEEKS, AS IT WAS ABOUT TO COLLAPSE.

FIG. 1.—EARTHQUAKE DAMAGE. SAN FRANCISCO POST OFFICE.

FIG. 2.—EARTHQUAKE DAMAGE, SAN FRANCISCO POST OFFICE, DUE TO SINKING OF GROUND.

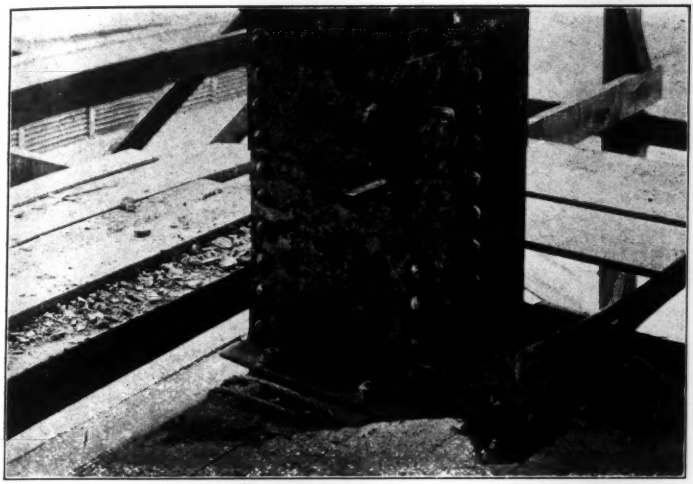


FIG. 1.—EARTHQUAKE DAMAGE, TOWER OF FERRY DEPOT. NOTE RUPTURED EYE OF DIAGONAL ROD.

FIG. 2.—FIRE DAMAGE, LATH AND PLASTER PARTITION, TENTH FLOOR, ATLAS BUILDING.

APPENDIX C.

REPORT OF COMMITTEE ON THE EFFECT OF THE EARTHQUAKE ON WATER-WORKS STRUCTURES.

COMMITTEE:
ARTHUR L. ADAMS, CHARLES D. MARX, CHARLES B. WING, C. E. MOORE, CHARLES GILMAN HYDE, C. E. GILMAN, GEORGE L. DILLMAN, FRANKLIN RIFFLE, PHILIP E. HARROUN.

The purpose of the investigation which has been made the basis of this report has been to ascertain the ability of the various classes of structures ordinarily used in the development and distribution of water supplies to withstand the violent shock of earthquake such as that which occurred in the coastal region of Central California on the morning of April 18th, 1906; and to draw such conclusions for improvement of design and for such increase of caution or confidence in the acceptance of established usages as the results seem to warrant.

In this report, no attempt will be made to describe in detail, or even mention, all the many structures appertaining to the various systems of works in use for supplying water within the great area violently shaken. The prime purpose will best be subserved by taking up the various most important classes of structures, and by setting forth, in the case of each, such information as the Committee has been able to obtain, and any conclusions which it feels warranted in drawing therefrom.

The classes of structures considered are: Earth Dams, Concrete Dams, Distributing Reservoirs, Pumping Stations, Elevated Tanks and Stand-pipes, and Pipe Lines.

The locations of the important structures mentioned hereinafter, and their position with reference to the geologic faulting where the movement originated, are indicated on Plate XL.

Earth Dams.—Centered about San Francisco there are several of the greatest earthen dams in the world. On the peninsula south of San Francisco, between San Francisco Bay and the Pacific Ocean, and immediately along the fault line or adjoining it, are three belonging to the Spring Valley Water Company and used as parts of the system supplying San Francisco with water. These are the Pilarcitos, the San Andreas, and the Upper Crystal Springs Dams.

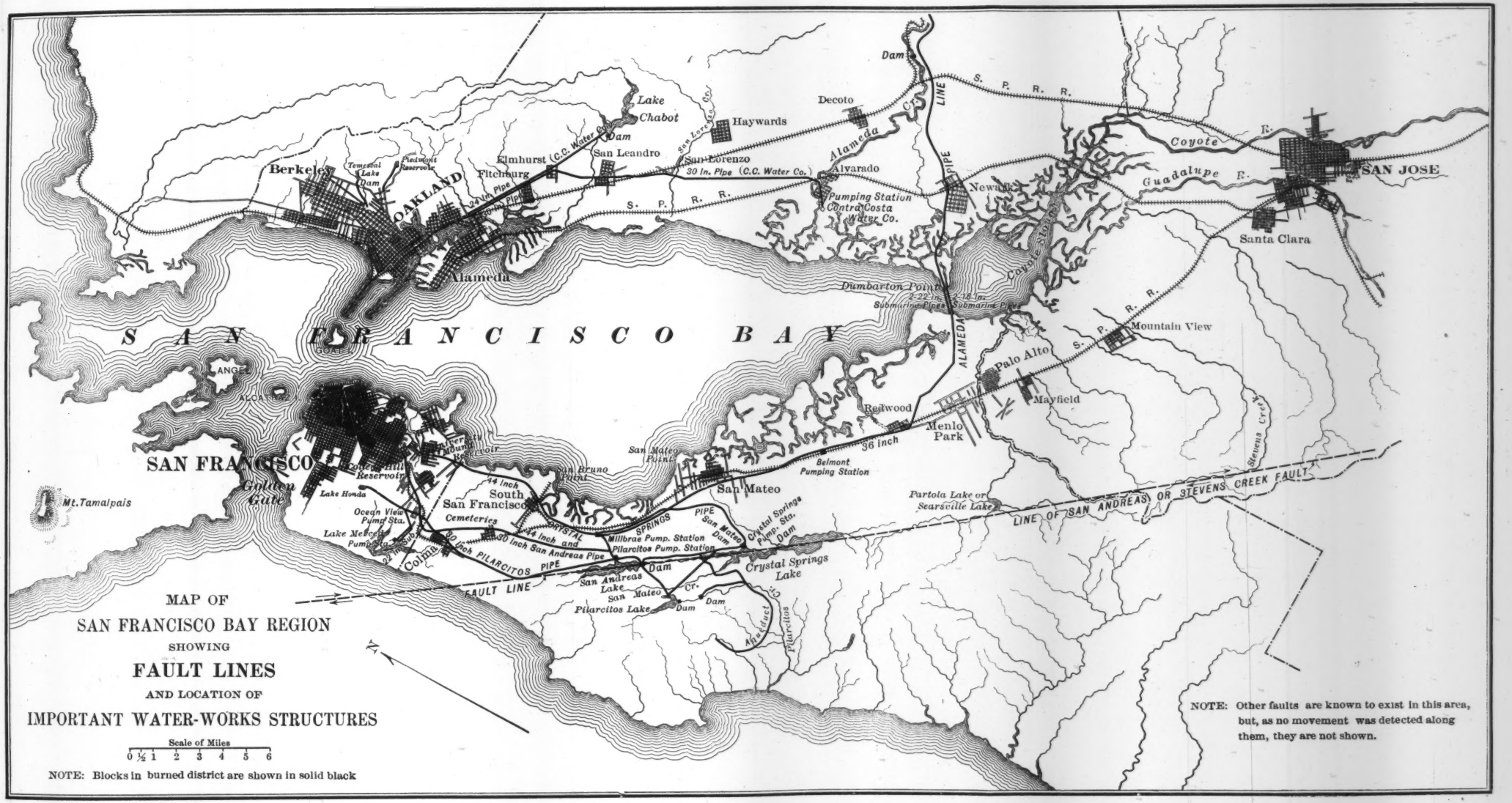
As the permanent longitudinal displacement along the fault line in this region is from 6 to 7 ft., all these dams must have

been shaken terrifically, especially the two which are directly on the fault line.

The Pilarcitos Dam is 640 ft. long on the top, and 95 ft. high; it has a top width of 24 ft. and slopes of 2 to 1 on both sides. It has a puddle core, 24 ft. thick, extending from 40 ft. below the bottom to the water level, and was built in 1864-66. It is 1¼ miles westward from the line of fracture. This dam was not injured.

The San Andreas Dam is 800 ft. long, 93 ft. high, and is of the same general type as the Pilarcitos Dam. It was built in 1868-70. The fault line passed across its east end, intersecting a natural ridge which was made a part of the dam at this point. The surface disturbance here was apparent over a width of about 150 ft. One of the cracks, without injury to the structure, intersected an inlet culvert built of concrete in which railroad iron was embedded. The brick-lined tunnel leading from the waste overflow was intersected by the fault line, and was fractured and offset in the general movement. The main body of the dam shows a crack, from 2 to 3 in. in width, extending longitudinally along the center line for the entire length of the dam. Some slight cracking in a direction transverse to the dam also occurred. No leakage through the dam resulted, and the earth structure may be regarded as wholly uninjured.

The Upper Crystal Springs Dam is about 75 ft. high, some 600 or 700 ft. long, and was built in 1878. It is of the same general type as the dams previously described. The original purpose of its construction is now changed, and at the time of the earthquake the water was at the same elevation on each side of it, so that it was not resisting any unbalanced pressure. This dam was crossed by the fault line at a point east of the center, and the two parts were offset about 6 or 7 ft. The fences on either side of the road passing along the top of the dam were straight prior to the disturbance. The top of the dam shows many longitudinal and transverse cracks. The former, while not continuous, appear along the entire length, being especially noticeable on the slopes. One is shown in Fig. 1, Plate XLI. Some years ago this dam was raised quite a number of feet, in order to improve the grades of the county road which traverses it. This work is said to have been done with much less care than would have been exercised had the upper part been intended for use as a dam. This condition, and the high line of saturation resulting from its being submerged on both slopes, are accountable, no doubt in large part, for the formation of longitudinal cracks through a tendency of the slopes to take a flatter inclination when subjected to a violent shaking. To what extent the ability of the structure to resist the pressure of



MAP OF
SAN FRANCISCO BAY REGION
SHOWING
FAULT LINES
AND LOCATION OF
IMPORTANT WATER-WORKS STRUCTURES

Scale of Miles
0 ½ 1 2 3 4 5 6

NOTE: Blocks in burned district are shown in solid black

NOTE: Other faults are known to exist in this area, but, as no movement was detected along them, they are not shown.

water has been affected cannot be determined under the circumstances; but the nature and extent of its apparent injuries are not so severe as to indicate that there would have been serious danger of failure had it been acting as a true dam in service.

In addition to these structures, much interest attaches to the two smaller dams which, by closing each end of a saddle depression, form the Saratoga Reservoir of the San José Water Company, located in the Santa Cruz Mountains between Saratoga and Los Gatos. The fault line runs directly through this reservoir, and crosses both dams approximately at right angles. Near the east end of the North Dam transverse cracks are found extending through the body of the dam. Fig. 2, Plate XLI, shows the crack on the wet slope of the North Dam. There was also a longitudinal crack through the dam, and some settlement of the inner slope. A transverse crack near the fault is shown in Fig. 1, Plate XLII. Though the reservoir was apparently full at the time, there is no evidence that any water escaped through the North Dam.

At the south end, the fault line crosses both the dam and a 10-in. cast-iron outlet pipe, which seems to have been shattered. A joint near the outer toe of the dam was also pulled apart. These pipe ruptures resulted in emptying the reservoir and washing out a considerable quantity of material from the South Dam, as shown in Fig. 2, Plate XLII.

On the eastern side of San Francisco Bay, in connection with the works of the Contra Costa Water Company, supplying the Cities of Oakland, Berkeley, and Alameda with water, are two very high earthen dams known as the San Leandro or Lake Chabot Dam and the Temescal Dam. In addition to these there is a third, of very recent construction, known as the Piedmont Dam.

The first of these dams was built in 1874-75, and is still the highest earthen dam in the world, the top being 127 ft. above the original surface on the center line. On April 18th the lake created by this dam was full to overflowing. The earthquake shock caused waves 3½ ft. high to break against the dam and pass over the spillways. Neither the dam nor any of its appurtenant structures were injured; nor were there left any evidences of its rough treatment.

The Temescal Dam is 105 ft. high, and was built in 1868. It was entirely uninjured.

The Piedmont Dam is a new structure, 260 ft. long on top, 45 ft. high at the inner toe, and 65 ft. high at the outer toe, with inner and outer slopes of 2 to 1. The inner slope was covered with 6 in. of concrete, with cement-plaster finish in lieu of the customary rip-rap, the concrete being placed in squares separated by asphaltum joints. The structure had been completed only a few months, and had just been filled for the first time. The rough

shaking it received had the effect of settling it about 6 in. at the center and producing a few small transverse and longitudinal cracks near one end of the dam. No cracking of the concrete, and no material injury of any character resulted.

All these dams were constructed by the ordinary method of spreading in thin layers, wetting, and rolling. Against the outer slopes of the San Leandro and Temescal Dams had been deposited, by sluicing, a large additional quantity of material. These two dams have puddle heart-walls. The Piedmont Dam is built without a heart-wall of any kind, but the better grade of material had been placed adjacent to the upper slope.

Concrete Dams.—The only large masonry dams in the region seriously shaken are the great concrete dam near San Mateo known as the San Mateo or Crystal Springs Dam, and the Partola or Searsville Dam. The former is a part of the system supplying San Francisco, and the latter is a part of the system supplying Stanford University. That at San Mateo is one of the great dams of the world, its projected height being 170 ft., with a finished width at top of 25 ft., and a thickness at base of 176 ft. Its present height is 146 ft. Its length when completed will be 680 ft.

The Partola Dam is a much smaller concrete structure, at present 50 ft. in height, though projected to a considerable additional height.

Both are built of irregular, interlocked blocks, formed in place, but are substantially monolithic. Each stands about parallel to the fault line and within a few hundred feet of it. Neither structure gives any evidence of injury in any particular. It is impossible to say how they would have been affected had they stood at right angles with the fault line and had they been intersected by it, as in the case of the earthen dams previously described. It seems reasonable to suppose that, had they been sheared through vertically and offset, as in the case of the earthen dams, or had they been torn up as to their foundations, nothing worse would have occurred than the gradual loss of the water from the reservoir, as both structures are designed with heavy gravity sections.

Distributing Reservoirs.—Of the numerous distributing reservoirs in use in the disturbed region, most of which are lined with brick, concrete, concrete and asphalt, or with clay puddle protected with paving stones, the Committee has heard of only one being injured. This was the reservoir in San Francisco known as Lake Honda, a structure holding about 33 000 000 gal. This structure was formed by an excavation in the bottom of a ravine, the hills rising all about it save at the outlet. The natural drainage is all diverted around it through a brick-lined tunnel. Either the reservoir has been dug to a depth well below the original natural sur-

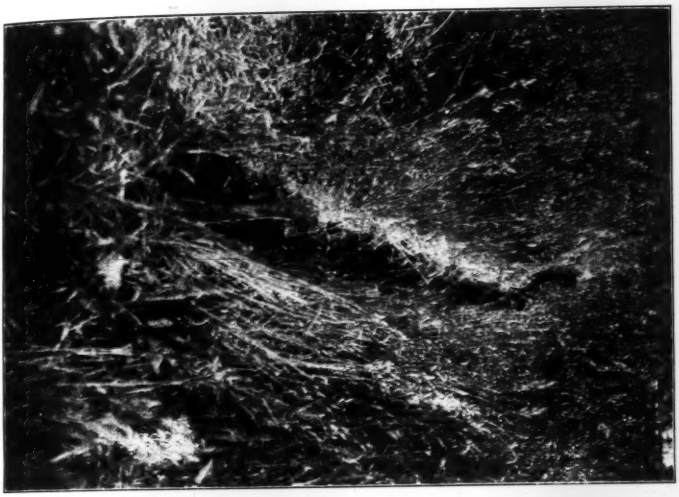


FIG. 1.—FISSURE IN SLOPE OF UPPER CRYSTAL SPRINGS DAM.

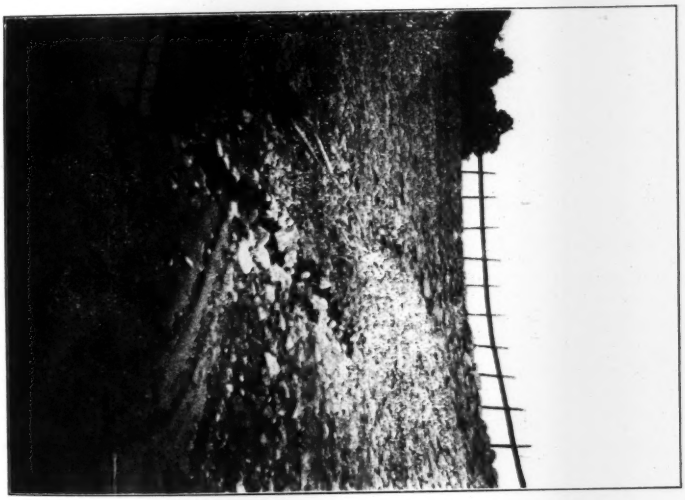


FIG. 2.—CRACKED AND DISPLACED RIP-RAP ON FAULT LINE AT DAM OF SAN JOSÉ WATER COMPANY NEAR SARATOGA.

face, even at the outlet, or there has been so great a volume of waste deposited below it that it is practically wholly in excavation, without banks on any side. The lining, consisting of masonry of various classes, was broken to an extent calling for repairs.

The slope, on the west side consisting of very sandy soil, was supported by a retaining wall which constituted the side of the reservoir basin. This wall consisted of a number of brick buttresses between which were horizontal arches, also of brick. The effect of the shock was to rupture this retaining wall, which failed by shearing near the bottom of the reservoir, although irregular cracks in one place extended to the top of the wall.

Pumping Stations.—On the peninsula between San Francisco Bay and the Pacific Ocean there are, in connection with the San Francisco Water-Works, six important pumping stations. Two of these are within the built-up sections of San Francisco, one is at Lake Merced, one at Milbrae, one at Belmont, and one near the San Mateo Dam. These stations are all equipped with water-tube boilers and horizontal, cross-compound, crank and fly-wheel, pumping engines of capacities (save for the last-mentioned plant) varying in different engines from 5 000 000 to 8 000 000 gal. in 24 hours. Each station contains a number of pumps; the Belmont Station, being the largest in total capacity, contains five pumping engines.

Save for the two stations in the built-up sections of San Francisco, individual, guyed, steel stacks were used; and the buildings were of corrugated-iron construction, supported on timber frames. Of the two plants in San Francisco, one was housed in a substantial brick building and the other in an old structure partly brick and partly corrugated iron; and both were supplied with brick stacks.

These pumping plants did not suffer any material damage affecting their serviceability, probably because their foundations were sufficiently massive to cause important units to move as a whole.

On the eastern side of San Francisco Bay there is an important pumping station from which about one-third of the water supply for the cities on that side is supplied. It is on the marsh land near the Bay shore, one mile west of Alvarado. It consists of two horizontal, cross-compound, crank and fly-wheel engines, each having a capacity of 5 000 000 gal. per day, and a full equipment of boilers, all housed in a brick structure. Individual, guyed, steel stacks are used. The engines, boilers, and building are supported on pile and concrete foundations of excellent construction. Although the brick walls of a part of the building were damaged so badly as to require reconstruction, the machinery equipment was not damaged in any particular.

The fuel-oil tank, having a capacity of 30 000 gal., holding at

the time about 20 000 gal., supported by a timber grillage resting on the surface of the ground at an elevation of about 3 ft. above the bed of a creek near by, was so violently shaken that several thousand gallons of oil were thrown out, the roof was thrown off, and the foundation settled about 2 ft., breaking all the pipe connections. During the quake the channel of the creek disappeared, its bottom being raised to the general level of the adjoining land.

Elevated Water-Tanks and Stand-Pipes.—At Milbrae and Belmont, on the western side of San Francisco Bay, there are two stand-pipes of considerable height and small section, stiffened for their entire height by an enclosing braced timber tower. These pipes are used, in lieu of relief valves, to control the pressures at the pumping stations of the Spring Valley Water Company at these places.

At Agnews and Santa Clara, south of the Bay, there were elevated tanks, almost, if not quite, identical in design. They consisted of four 45 000-gal. wooden tanks supported on a steel frame 80 ft. in height.

There were also in the disturbed area many other structures of less magnitude, consisting of wooden tanks supported on timber towers.

Of the structures connected with public water-supply works, of sufficient importance to warrant special mention, only one was wrecked, although many low tanks used for road sprinkling, and others for locomotive water supply on the railroads, were thrown down. This wrecked tank was the one at Santa Clara, previously mentioned. Plates XLIII and XLIV show its appearance during construction and before and after the failure. Concerning this structure, the designing engineer, C. E. Moore, M. Am. Soc. C. E., residing at Santa Clara, says:

"At Santa Clara the shake was especially violent at the pumping station; and near here the 12-in. supply pipe was broken, evidently by the upheaval. The pipe was shattered at the top and the broken bottom so telescoped as to produce an overlap of 1½ in. There is very decided evidence of wave action and consequent vertical motion. The 10-in. stand-pipe leading to the tanks was lifted out of its connection with the main, stripping the threads connecting it to a cast-iron flange.

"The water-tower itself was wrecked, but in such a manner as to testify strongly to the ability of a steel frame to withstand tremendous wrenches.

"The tower was of steel, 80 ft. high, and was made up of sixteen Z-bar columns, four rows of four each, carrying four rows of plate girders. Resting on the plate girders were 15-in. I-beams forming the floor system. The load consisted of four tanks with a total capacity of 180 000 gal. At the time of the disaster, there were in the tanks about 125 000 gal. As gathered from those who

FIG. 1.—TRANSVERSE CRACK IN DAM OF SAN JOSÉ WATER COMPANY NEAR SARATOGA. THIS CRACK IS ALONG THE MAIN FAULT LINE.

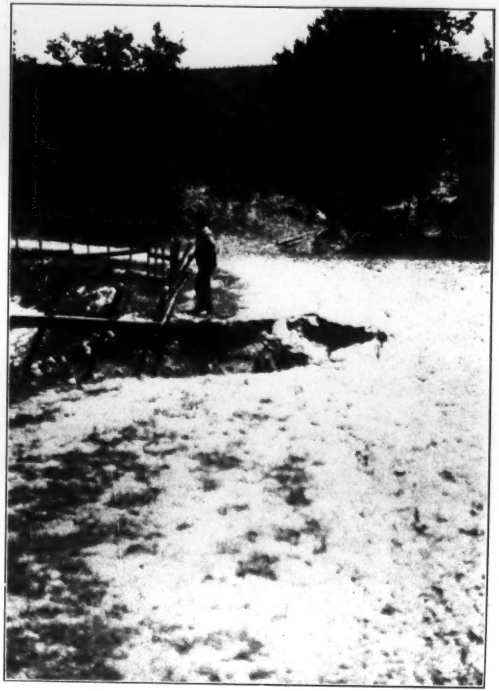


FIG. 2.—HOLE IN DAM OF SAN JOSÉ WATER COMPANY NEAR SARATOGA. THIS HOLE IS NEAR THE MAIN FAULT LINE.

observed this structure at the time, and from the position of the parts, the action was about as follows:

"The structure swayed violently. At the same time the tanks lifted from their setting and moved toward the south in a series of lurches. Finally, the two south tanks shot off from the floor and struck the ground, about 75 ft. from the tower. The two north tanks had moved to the extreme southern portion of the floor. The earthquake was then practically over. The structure ceased to sway, and stood upright. The movement of the tanks, however, had turned over some of the floor beams, and, being unable to carry the load in this position, they were bent downward. Their ends slipped from their support, letting fall the two remaining tanks, which in falling wrecked the tower.

"The tanks carried with them the lateral bracing, and pulled down the columns. On the north side many of the columns still stand intact with all lateral and sway-bracing uninjured, excepting the top story, which was pulled down by the girders.

"The fall took place several seconds after the shake was over; and the tanks seemed to the observers to settle slowly through the floors.

"The structure will probably be rebuilt along the original lines, as much of the material is uninjured. Additional bracing, however, will be placed between the girders to resist overturning; and, in place of the four wooden tanks, will be substituted one large steel tank anchored to the I-beams.

"* * * Had these features been embodied in the original design, I am of the opinion that the structure would now be standing. The possibility of such movement in the tanks, however, was never contemplated in the preparation of the plans."

The lesson to be drawn from this failure is evidently the necessity for thorough floor bracing and the secure anchoring of the load, as Mr. Moore has pointed out. With such precautions, it seems evident that such structures can more successfully resist earthquake shock than can most buildings.

Pipe Lines.—The effect of the earthquake upon pipe lines and conduits generally depends:

First, upon the character of the foundation in which the structure is placed, as to whether it is firm and unyielding or such as will settle or displace seriously under violent agitation, such for instance as marsh, filled, or any soft alluvial soil;

Second, upon the character of the structures supporting the conduit if laid above ground; and

Third, on whether the structure is intersected by the fault line.

As far as the Committee has been able to ascertain, no considerable damage resulted to any pipe line which had been laid in firm ground.

All important failures occurred either where they were supported above the ground on wooden trestles; where the shock produced serious and unequal settlement; where subject to the suspicion of having been strained from other causes; or where intersected by the fault line.

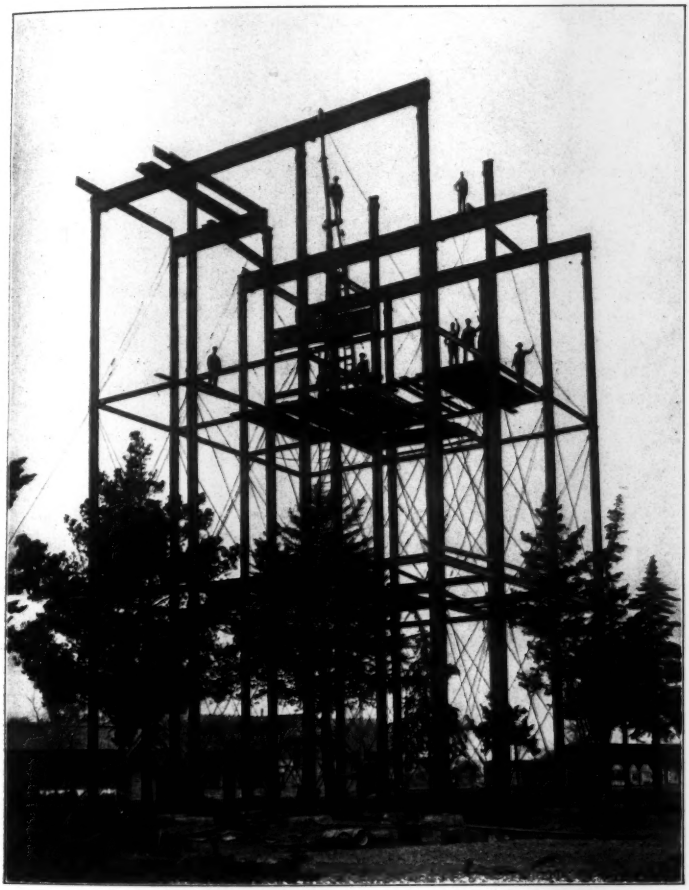
Of the three, long, riveted-iron, pipe lines leading from storage reservoirs into San Francisco, the middle one, 36 in. in diameter, leading from San Andreas Reservoir, was fractured at one point only. This was at Baden, where the pipe crossed a piece of marsh land on a wooden trestle. A slip joint had been provided at this point, and the amount of movement was restricted by four bolts attached to lugs riveted to the pipe on each side of the joint. The ¾-in. sheet to which these lugs were attached was torn out. Fig. 1, Plate XLV, shows the location of the break and the appearance of the pipe after temporary repairs had been made.

The 44-in. pipe leading from the San Mateo or Crystal Springs Reservoir into San Francisco had three breaks known to the Committee. Two of these were not serious. The third was the most extensive which occurred at any point about the Bay, save those immediately on the fault line hereinafter mentioned. This was at the crossing of the San Bruno Marsh, where, for a distance of about 2 000 ft., the pipe was supported upon trestle bents resting on a pile foundation. The pipe was pulled apart at a number of places, and nearly all of it was thrown entirely off the supports on either side. The photograph, Fig. 2, Plate XLV, taken after making repairs to the pipe, shows the location, and the character of the work during reconstruction.

The trestle supporting the pipe was no doubt still substantial enough to meet all conditions other than the unusual one by reason of which it was destroyed; but many of the timbers were badly decayed, and it is possible that this contributed to its destruction. This possibility is supported by the fact that no breaks have been observed at other points where this same line, supported on trestles, crosses marsh land. In these instances, the trestles were in good condition, but were not nearly as long as at the San Bruno crossing.

The 12-in. cast-iron pipe, 5 miles long, leading from the Searsville Reservoir to Stanford University, was cracked in three places. All occurred on horizontal curves. One was a fractured bell, and the other two presented longitudinal cracks, 4 or 5 ft. long. None was accompanied with evidence of earth displacement.

The pipe line leading from the Pilarcitos Reservoir to Lake Honda in San Francisco is, in small part, of 24-in. cast iron and 22-in. riveted iron, almost all of it being 30-in. riveted iron. It is located for some miles along the route of the ancient geologic fault on which the movement of April 18th occurred, and is



WATER TOWER, SANTA CLARA WATER-WORKS, IN PROCESS OF ERECTION.

crossed and recrossed by it many times. The permanent longitudinal movement was in general from 6 to 7 ft., and, wherever crossed by the fault line, the pipe, whether of cast or riveted iron, whether buried in the earth or carried on a trestle, was badly shattered, the direction of crossing determining whether the pipe was torn apart or telescoped. Figs. 1 and 2, Plate XLVI, illustrate typical fractures. At one point a trestle, crossing at a point where the pipe departs some distance from the fault line, was completely destroyed, the entire trestle and pipe line being thrown clear off its original site. Figs. 1 and 2, Plate XLVII, show quite clearly the appearance of the surface fracture of the ground along the line of movement in the immediate vicinity of the pipe line, it being discernible in the latter picture as far as the horizon. Fig. 1, Plate XLIX, shows by the offset in the fence the 6½ ft. of longitudinal displacement.

It is evident enough that no precaution can prevent the failure of a structure where crossed by a fault line in the event of a movement as great as that which here occurred.

In San Francisco, all serious fractures of water mains, as a result of the earthquake, were due to lateral displacements, or subsidences of filled or soft ground across which, unfortunately, the main supply pipes from the lower and middle service distributing reservoirs passed. The map, Plate XLVIII, shows these areas bordered with a heavy line where clearly defined and with a broken line where not so distinctly and connectedly traceable. The displacement laterally amounted in places to as much as 6 or 7 ft.; vertically, it amounted to several feet. Fig. 2, Plate XLIX, gives some idea of the extent of this movement, both laterally and vertically, at Valencia and 18th Streets, where one of the most serious breaks occurred. It also shows the pipes laid temporarily on the surface to replace those parted underground.

It is evident that no precautions, save the entire avoidance of such localities in selecting the routes of main lines, can safeguard a pipe system against almost instant disability in event of causes leading to displacements of such magnitude. If entire avoidance is not possible, in some cases partial or entire immunity from risk might be found in supporting the pipes on deep pile foundations extending well below the compressible soil, and in the use of heavy flexible joints across lines of probable separation.

Extending from Bakersfield, in the south central part of the State, to Point Richmond, on the east side of San Francisco Bay, the Standard Oil Company has an extra heavy 8-in. steel screw pipe for conducting oil, by relay pumping, a distance of 300 miles. Mr. Page, the engineer in charge, reports the line absolutely uninjured.

On the pipe lines of the Contra Costa Water Company, supplying the cities on the eastern side of San Francisco Bay, a few ruptures occurred, all of which were in marshy or low-lying alluvial soil. Because of a fortunate duplication of important pipes, and an ample storage at Lake Chabot, none was so serious as to interfere with the continuity of the service, to any section of the region supplied, for more than 2 or 3 hours.

At Alvarado, near the pumping station, the 30-in. riveted-iron force main conducting the water to Oakland crosses the Alameda Creek on a short bridge. At a point just above the ground, at one end of the bridge, the pipe was pulled apart by the shock, and the two ends jammed against one another three times by the succeeding vibrations, the abutting edges of the sheets cutting into each other at different places each time a total distance of 5 in. The photographs of the two courses taken after their removal, Figs. 1 and 2, Plate L, show the cut edges on opposite sides of the pipe. A short distance from this point, a renewal of the 30-in. pipe had in recent years been made with 24-in. pipe supported for one mile on two-pile bents spaced 29 ft. between centers. In this pipe was placed a 24-in. stop-gate resting on a concrete foundation. The piles under the pipe were not deeply placed; and the shake operated to settle them irregularly, some going down as much as 6 in. This did not damage the pipe, but brought a strain on the gate which broke off the flange.

A 24-in. riveted pipe lying in a street leading across a tide marsh in Oakland was pulled apart 5 in. and displaced 8 in. laterally by the settling of the entire street.

One 12-in. cast-iron pipe near the water front in East Oakland was drawn apart at the joint, and one or two breakages occurred on a long line of 8-in. pipe leading to the Southern Pacific Railroad Broad-Gauge Pier.

These were all the breaks that occurred on the several hundred miles of pipe in use on the east side of San Francisco Bay.

Conclusions to be Drawn.

The conclusions of the Committee are as follows:

First.—That greater attention in the future should be given to the avoidance, when possible, of threatening geologic faults, in the location of important water-works structures, than has sometimes been given in the past.

Second.—That skillfully designed and well-built earthen dams are structures proven by the great earthquake of April 18th, 1906, to be of great stability, and deserving of increased confidence.

Third.—That concrete dams of gravity section are able to withstand earthquake shocks of great severity without damage.

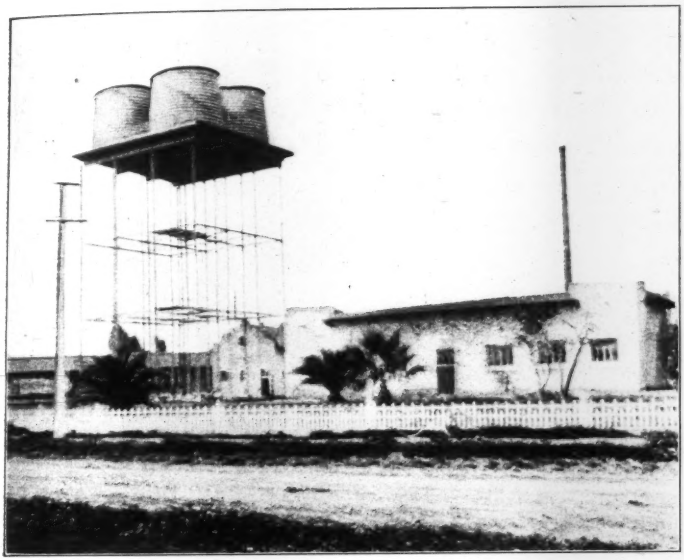


FIG. 1.—WATER TOWER, SANTA CLARA WATER-WORKS, BEFORE ITS DESTRUCTION.

FIG. 2.—WATER TOWER, SANTA CLARA WATER-WORKS, AFTER ITS DESTRUCTION, APRIL 18TH, 1906.

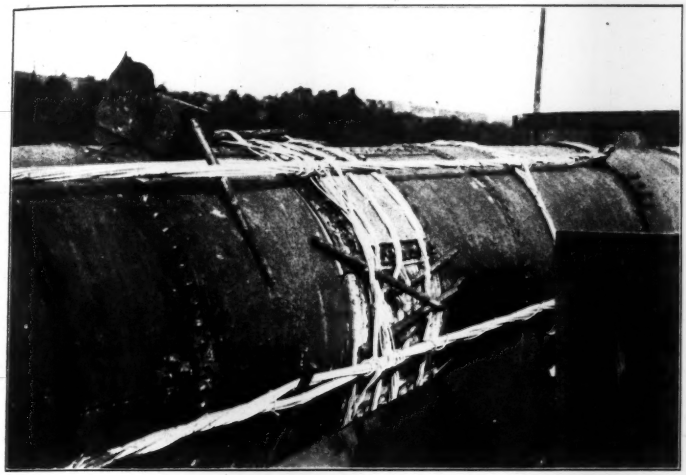


FIG. 1.—SAN ANDREAS 36-IN. PIPE LINE NEAR BADEN, AS TEMPORARILY REPAIRED.

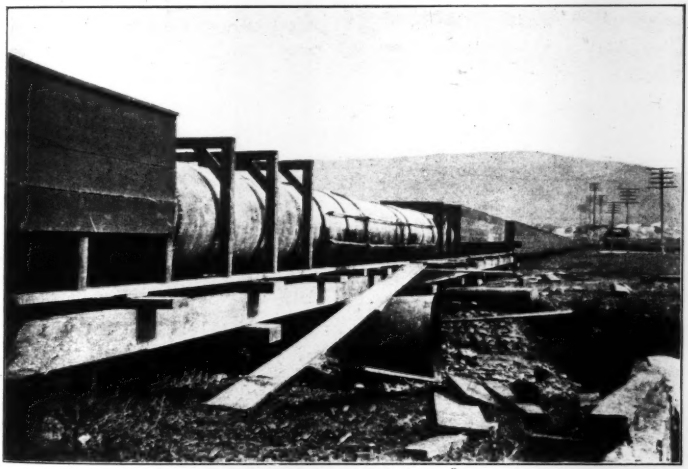


FIG. 2.—CRYSTAL SPRINGS 44-IN. PIPE LINE CROSSING SAN BRUNO MARSH.

FIG. 1.—BREAK IN PILARCITOS PIPE LINE.

FIG. 2.—BREAK IN PILARCITOS PIPE LINE.

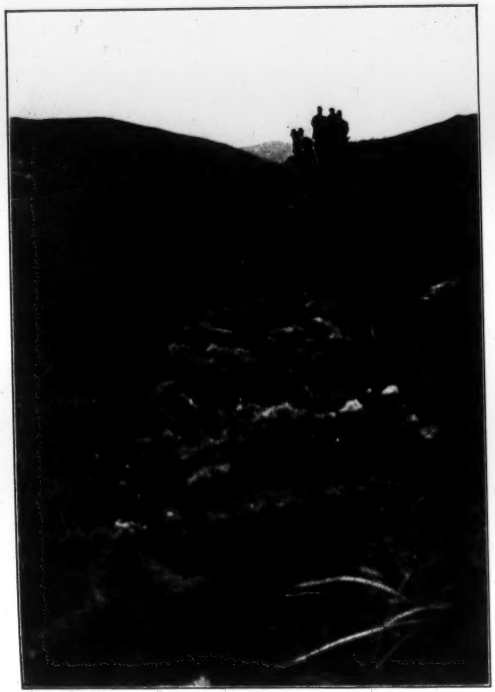


FIG. 1.—SURFACE INDICATIONS OF FAULT LINE ALONG THE PILARCITOS PIPE LINE.

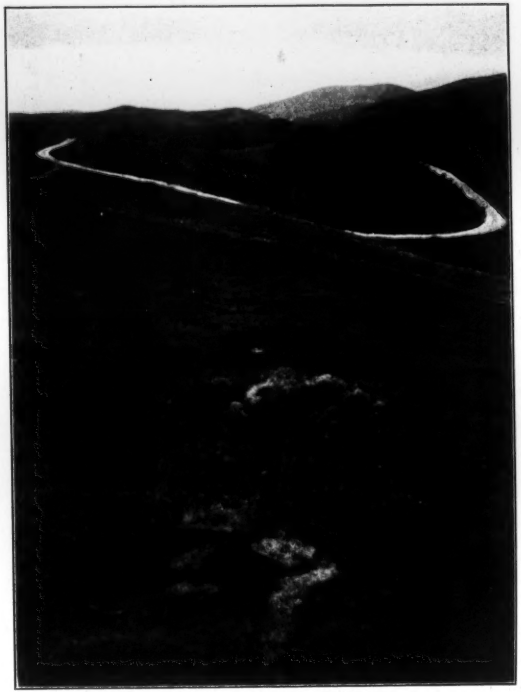
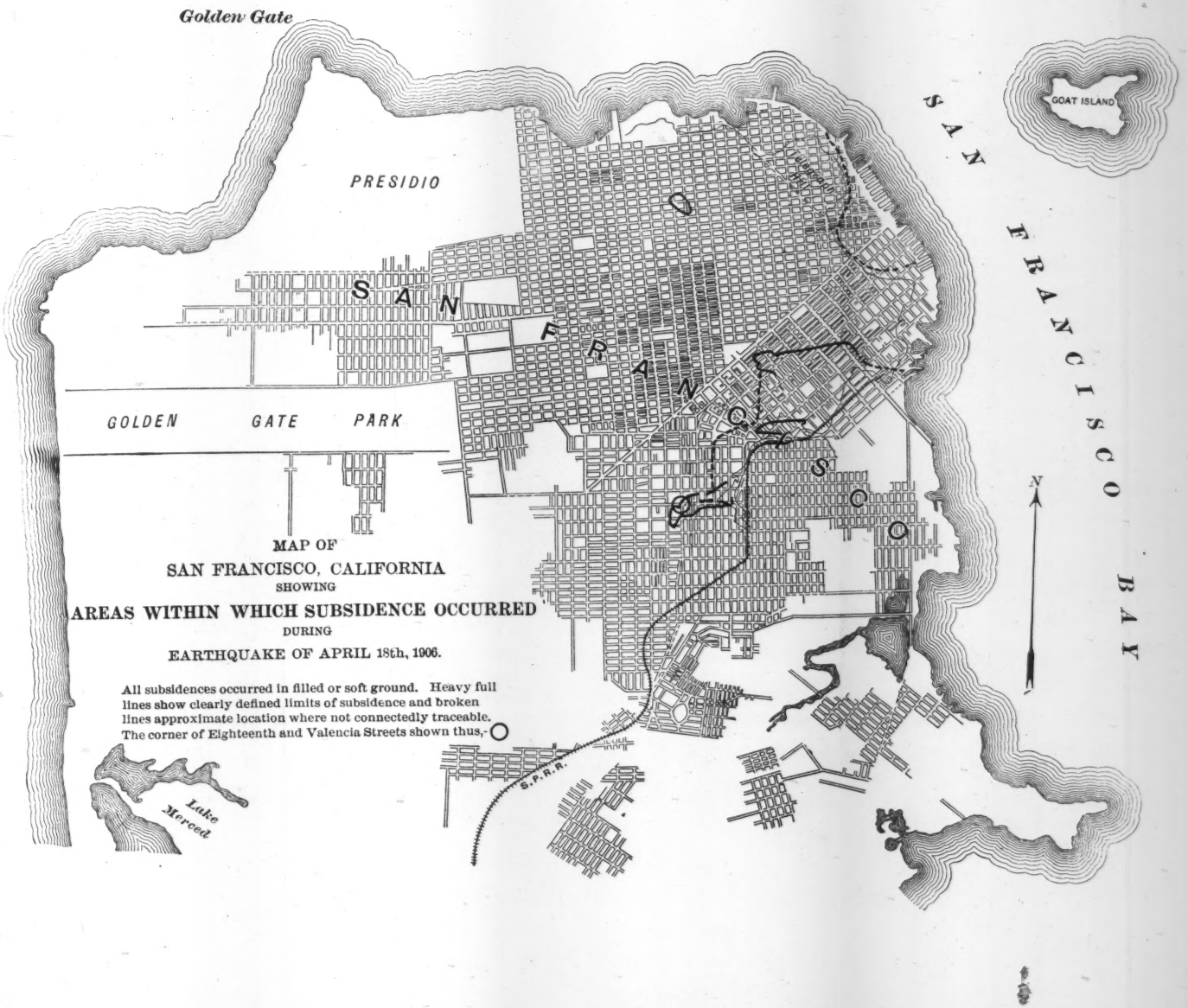


FIG. 2.—SURFACE INDICATIONS OF FAULT LINE ALONG THE PILARCITOS PIPE LINE.

PLATE XLVIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LIX, No. 1056.
SAN FRANCISCO EARTHQUAKE.
REPORT ON WATER-WORKS.



MAP OF
SAN FRANCISCO, CALIFORNIA
SHOWING
AREAS WITHIN WHICH SUBSIDENCE OCCURRED
DURING
EARTHQUAKE OF APRIL 18th, 1906.

All subsidences occurred in filled or soft ground. Heavy full lines show clearly defined limits of subsidence and broken lines approximate location where not connectedly traceable. The corner of Eighteenth and Valencia Streets shown thus,-O

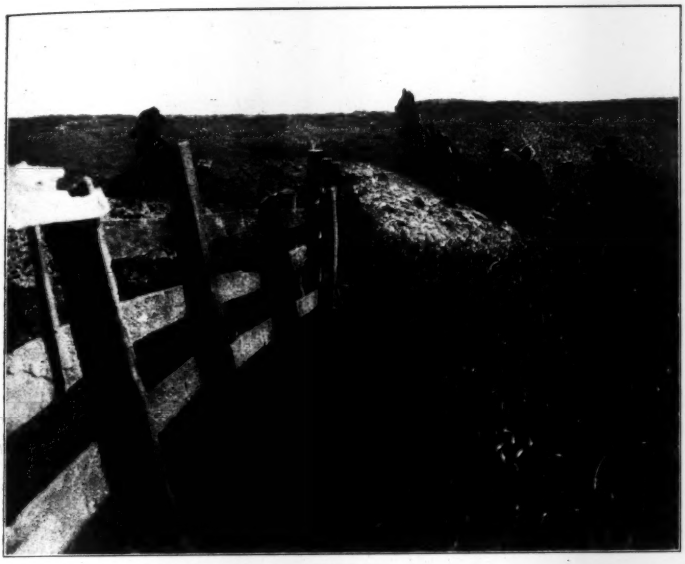


FIG. 1.—LONGITUDINAL DISPLACEMENT ON THE FAULT LINE, AS SHOWN BY A FENCE AT A PLACE INTERSECTING THE PILARCITOS PIPE LINE.

FIG. 2.—DISPLACEMENT OF GROUND AT EIGHTEENTH AND VALENCIA STREETS, SAN FRANCISCO.

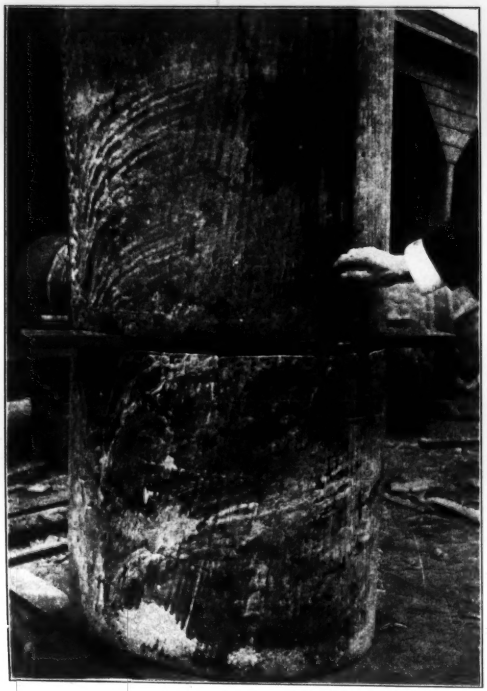


FIG. 1.—SECTIONS FROM A 30-IN. PIPE LINE NEAR ALVARADO, SHOWING CUTS MADE BY THREE SUCCESSIVE EARTHQUAKE SHOCKS.

FIG. 2.—OPPOSITE SIDE OF PIPE SECTION SHOWN BY FIG. 1.

Fourth.—That distributing reservoirs, pumping machinery, elevated tanks and stand-pipes, if made secure as to foundations, and intelligently designed along lines of the best accepted practice, will be found capable of withstanding earthquake shocks sufficiently severe to work the general destruction of most buildings.

Fifth.—That pipe lines or conduits, of any character whatever, are almost certain to fail when intersected by a plane of large movement, whether that movement be in the nature of a faulting, or a sliding or settling of the earth, in consequence of earthquake shock; and that, in choosing locations for such structures, it is prudent, as far as practicable, to avoid localities subject to such movements.

That in planning distributing systems for cities, the important supply pipes should be located along routes carefully selected for stability; and that areas liable to serious disturbance should be so segregated in the piping system as to make them easily and quickly separable from the remainder of the system.

Sixth.—That close adherence should be given to the recognized standards of the best present practice, such as the use in distributing systems of an abundance of suitably placed gates for isolating different pipes and different sections; the preservation of such gates in working condition; and having their location easily ascertainable, both in the street and in the office records; the duplication, along widely separated routes, of the critically important main supply pipes; the maintenance in as close proximity as may be to the centers of use of an ample storage supply of water; and the use of types of substantial design in working out plans for structures subject to possible earthquake shock.

In conclusion, the Committee gratefully acknowledges its obligation to those who have rendered to it valuable aid, and particularly to the officials of the corporations controlling the water-works, for facilities for making inspections.*

*This report was accompanied by 29 photographs, from which 17 have been selected for reproduction. They are shown in Plates XL to L.

APPENDIX D.

REPORT OF COMMITTEE ON THE EFFECT OF THE EARTHQUAKE ON LOCAL TRANSPORTATION AND LIGHTING PLANTS.

COMMITTEE:
A. M. HUNT.

Street-Car Tracks.—The injury caused to the street-car tracks was relatively slight. Where the tracks crossed the areas within which there was actual displacement of the ground, they were moved bodily, and there was serious sinking. Wherever this occurred, it was on ground that had been filled.

The slots of cable roads were carefully inspected after the fire, and in numerous places in the burned district were found closed. A characteristic closure of the cable slots is shown by Plate LI. Except within the areas of slips, above noted, these points, with few exceptions, were found only in the burned district, and the evidence points to its being due mainly to expansion caused by the excessive heat. The fact that the Geary Street line, which is a cable road, operated for several hours after the earthquake, is very pertinent.

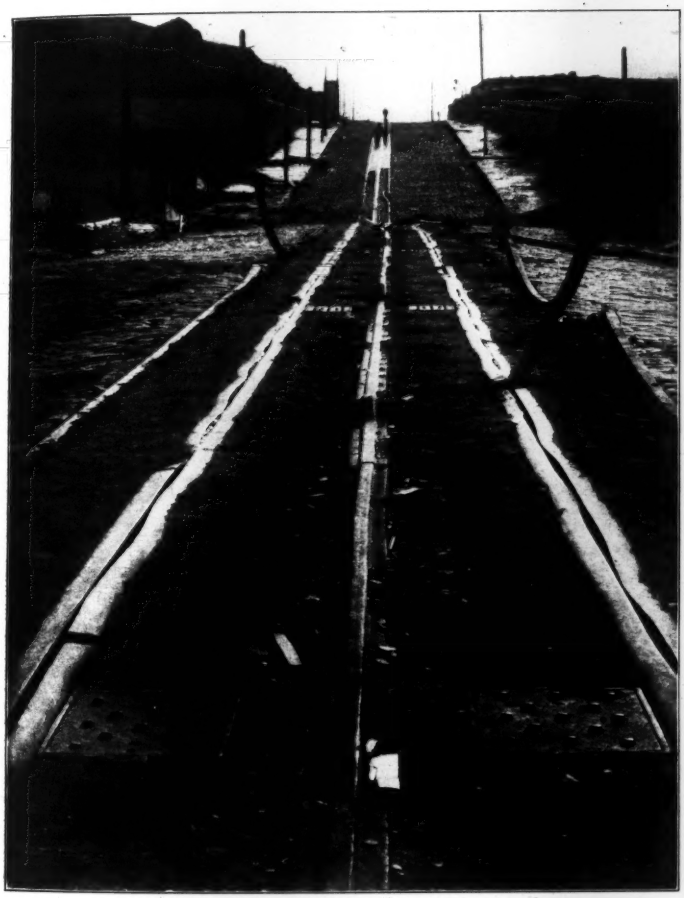
In the opinion of the Committee, and judging from the available evidence, any of the cable lines, except the Valencia Street line, could have been put in condition to operate, as far as the cable slot and conduit are concerned, in a few days, had not the fire followed the shake. The result of the fire and the falling debris was to put them entirely out of business, and make the amount of work necessary to put them into shape very serious.

Many of the trolley lines outside of the burned district were in such shape that they could have been operated at any time, but, owing to the fire hazard from the tangle of other wires, and the lack of water in many districts, they were not started for some days.

The trolley lines unquestionably substantiate the claim that such systems are flexible, and admit of emergency work to an extent not equalled by any others.

The Bryant Street power-house of the United Railroads was practically uninjured. The North Beach plant of the same company was temporarily crippled by the brick stack being shaken off and the roof collapsing.

Gas Plants.—Immediately after the earthquake, all gas was shut off the city lines, but sufficient gas was in the mains to cause



CHARACTERISTIC CLOSURE OF CABLE SLOTS IN BURNED DISTRICT OF SAN FRANCISCO. THE VIEW ALSO SHOWS BROKEN CAST-WELD JOINTS, AND EXTREME WARPING OF RAILS.

numerous explosions as the fire progressed. With the exception of the North Beach plant, the various gas-works were uninjured. The damage to mains was principally due to explosions.

As service was gradually restored, it was found that the house pipes, except in very few instances, were not damaged.

Electric Light Plant.—The electric lighting plant of the San Francisco Gas and Electric Company suffered very little injury by the earthquake, but the fire destroyed all the distributing substations, and played havoc with the overhead distributing lines. The underground conduit system and the Edison tubing came through both earthquake and fire in very good shape.

Miscellaneous.—Brick stacks of power plants were in most instances broken off, while well-guyed, steel stacks stood without injury.

APPENDIX E.

REPORT OF COMMITTEE ON THE EFFECT OF THE EARTHQUAKE ON RAILWAY STRUCTURES.

COMMITTEE:
J. H. WALLACE, H. C. PHILLIPS, R. M. DRAKE, E. M. BOGGS.

Within the area of destructive effect of the earthquake of April 18th, 1906, the following railroad companies are operating: Southern Pacific, Atchison, Topeka and Santa Fé, California Northwestern, and North Shore.

In common with other property owners, they all suffered more or less from the falling of chimneys and cornices, the cracking of brick walls and the falling of old and weak buildings.

The structures peculiar to railroads, which were affected, are embankments, trestles, bridges, tunnels, and water and oil tanks.

Embankments.—Embankments across marshes, or with soft strata underlying them, settled more or less. In some cases the settlement was vertical; in other cases there was considerable horizontal with the vertical movement.

At one point on the marsh between Benicia and Suisun, on the Southern Pacific, the settlement was 11 ft.; at another point, 5 ft. These were nearly vertical.

Between Niles and San José, on the Southern Pacific, there was at one point a displacement of 3 ft. horizontal, but the vertical displacement was only 6 in.

On the North Shore, about 2 miles north of Point Reyes, the road, originally, had been constructed with pile trestles across several arms of Tomales Bay; these trestles had been filled about 15 years ago, the road-bed being about 8 ft. above ordinary high tide. Two of these embankments, 2 200 ft. and 900 ft. long, respectively, sank until the water at high tide washed over the rails.

Settlements of this character, and of more or less severity, were common on all railroads operating about the Bay of San Francisco and its tributaries.

Trestles.—Trestles over marshes suffered more or less from the movement of the soft material into which the piles were driven. The most serious damage was at Fallons, on the North Shore, where a trestle, 600 ft. long and 70 ft. high, was thrown down. This trestle was constructed of framed bents on piles. Portions of the trestle over Lagunitas Creek, about a mile from Point Reyes, were thrown entirely off the piles, the piles themselves being moved down stream.

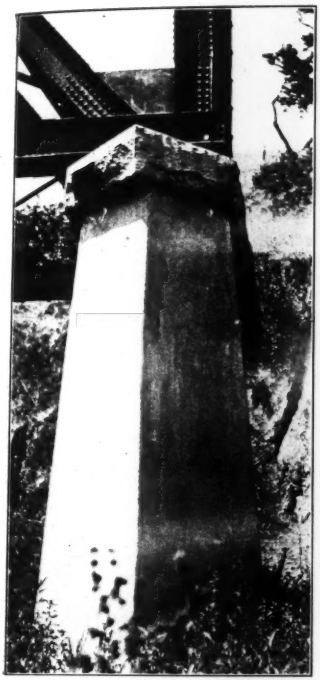


FIG. 1.—PIER 2, PAJARO RIVER BRIDGE.

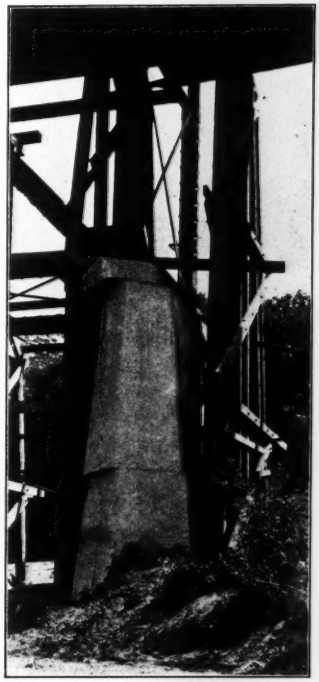


FIG. 2.—PIER 5, PAJARO RIVER BRIDGE.

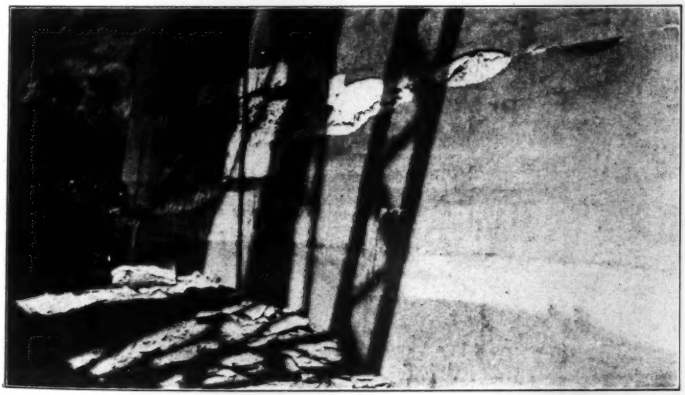


FIG. 3.—BASE OF PIER 3, PAJARO RIVER BRIDGE.

On the other roads the effect on trestles was very slight.

Draw-Bridges.—Draw-bridges across the little creeks and inlets around San Francisco Bay, being generally on soft ground, were affected by a slight movement of their piers, in many cases resulting in the bridge binding so that it could not be opened until some repairs were made.

The draw-bridge at Black Point, over Petaluma Creek, on the Sonoma Branch of the California Northwestern, was open at the time of the earthquake, and was thrown off its center 2 ft. to the east and 1 ft. to the north. This is a steel structure, 220 ft. long, on four iron caissons, filled with concrete, on pile foundations.

Fixed Bridges.—With a few exceptions, fixed bridges were not affected seriously. The bridges over the Russian River, at Healdsburg, and at Bohemia, on the California Northwestern, were both shifted slightly on the piers at one end.

At Duncan Mills, on the North Shore, a combination span, 120 ft. long, had the eye-bars in the lower chord buckled by movement of the abutments toward each other.

The Southern Pacific Bridge across the Pajaro River, at Watsonville, consisting of four 80-ft. wooden spans on pile piers, had the second pier from the east end moved (up stream) about 3 ft.

The bridge across the Pajaro River, near Chittenden, on the Southern Pacific, was badly damaged. The "line of fault" crosses this bridge near the west end. The bridge consists of three 120-ft. riveted spans with 50-ft. girders at each end. The general effect of the movement of the earth along the fault line was to increase the distance between the east and west abutments by 3.5 ft. The shock affected all piers and abutments more or less. Fig. 3 is a plan showing the original position of this bridge and the displacement of the piers; and Figs. 1, 2, and 3, Plate LII, are photographs of some of the piers.

At the east abutment, or Pier 1, the wing-walls cracked and settled about 1½ in. At Pier 2, the coping stone was displaced. Pier 3 was moved eastward from its original position 0.55 ft.; the north end settled 4 in., and the south end was raised 2 in. The coping stone was loosened. This pier cracked in two places, the cracks being horizontal; they probably occurred along the line marking the end of a day's work.

Pier 4 was moved westward 0.8 ft. at its center, and was twisted slightly, the movement being a little more at the south than at the north end. It was raised about 0.16 ft. at the south end and about 0.02 ft. at the north end. This pier has a horizontal crack, probably on the line marking the end of a day's work. The portion above the crack moved, with reference to the portion below it, eastward 0.16 ft. The lower portion of the pier actually moved westward 0.8 ft.,

SKETCH SHOWING DISPLACEMENT OF PIERS OF THE PAJARO RIVER BRIDGE
AFTER THE EARTHQUAKE OF APRIL 18TH 1906,

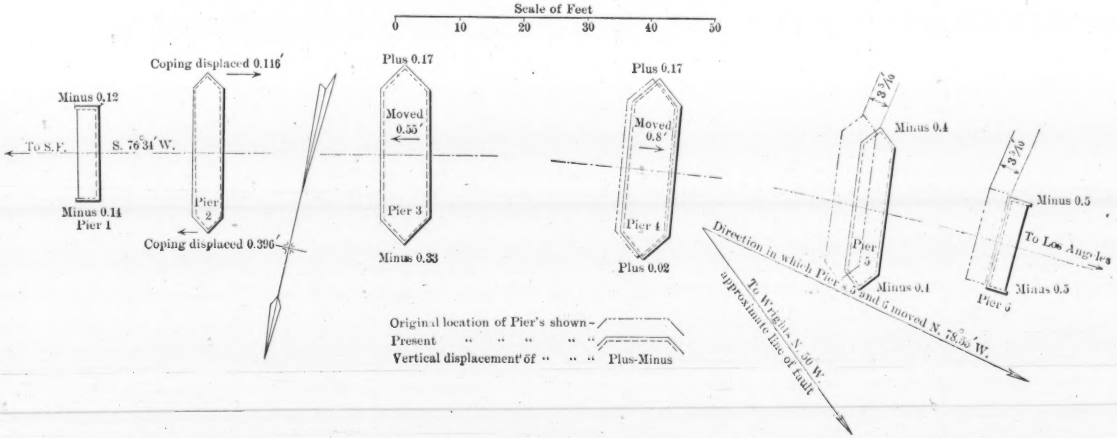


FIG. 3.

FIG. 1.—BRIDGE 39, NORTH SHORE RAILWAY, AFTER THE EARTHQUAKE.

FIG. 2.—PASSENGER TRAIN AT PT. REYES, NORTH SHORE RAILWAY, OVERTURNED BY THE EARTHQUAKE.

as noted above. The coping stone was loosened but not displaced to any extent.

The base of Pier 5 was moved westward 3.5 ft., and settled 0.4 ft. The pier broke along a horizontal line close to the ground surface, and the upper portion was moved eastward, with reference to the lower portion, 0.25 ft. The coping was torn from the top of the pier, being dragged by the end of the span anchored to it about 2 ft. from its proper position.

Pier 6, or the west abutment, was moved 3.5 ft. eastward, but does not appear to be damaged.

The end of the 50-ft. girder which landed on this abutment was dragged off the bridge seat, but did not fall, being held up by the fastenings of the rail and the riveted connections to the next span to the east.

Piers 1, 2 and 3 are founded on shale; Pier 4 is founded on sandstone at the south end and on shale at the north end, the greater part of the pier being on shale. Piers 5 and 6 are on sandstone.

From this bridge to the line of fault in Wright's Tunnel, the course is N. 50° W., which corresponds with the direction of the line of fault in the tunnel at Wright's.

The bridge was built in 1900, replacing a wooden bridge. In 1890 there was an earthquake which threw the bridge out of line, one end being shifted, with reference to the other, about 1 ft.

Tunnels.—In general, tunnels seem to have been affected only by the displacement or loosening of the material in the sides and roof, caused by the shaking of the ground. The effect of this was to crush the timbers.

The new tunnels being constructed by the Southern Pacific Railroad on the Bay Shore line, in the southern part of San Francisco, were not injured at all.

The tunnel at Wright's Station, on the South Pacific Coast Railway (Southern Pacific Company), was on the line of the fault, and suffered considerable damage. The tunnel is 6 225 ft. long. The line of fault crosses the tunnel about 400 ft. from the north portal; the movement at this point was 5 ft., practically at right angles to the axis of the tunnel. The direction of the line of fault is N. 50° W.

In addition to this slip along the fault line, the timbers are broken, and rock and earth caved in from the roof and sides at a number of points.

The next tunnel south, which is 5 720 ft. long, was also badly damaged, the damage, however, being confined entirely to broken timbers and caves from the roof. On the morning of April 18th it was possible to walk through this tunnel, but the minor shocks

that occurred during the day shook down enough material to block it completely at four points, each ranging from 15 to 40 ft. in length. In addition to these four large caves, 150 sets of timbers were broken and scattered throughout the entire length of the tunnel.

The other tunnels along this line, between Glenwood and Santa Cruz, were not damaged to any extent. In one of them one set of timbers was broken, in another two sets were broken.

Tanks.—Several tanks on the different roads were thrown down. In all cases, however, these were on wooden or cast-iron frames, not braced to stand the twisting motion which accompanied this earthquake. Tanks on riveted frames did not fall, though in several instances they were severely damaged and thrown out of plumb.

Overturned Cars.—At Chittenden Station, on the Southern Pacific, three box-cars, two of them loaded, standing on the side track, were turned over. The track at this point runs N. 51° W., and the cars were turned over to the right side of the track.

On the North Shore (3-ft. gauge), a passenger train consisting of three cars and an engine, standing on the track at Point Reyes, was thrown over on its side, as shown by Fig. 2, Plate LIII.

On the Southern Pacific, a freight train which was running at the time toward San Francisco, a few miles south of Chittenden, was derailed.

In the mountain region adjoining the line of fault there were a great many slides of earth and loose rock. When these occurred in railroad cuts there was some little delay of traffic, but the actual damage was small.

APPENDIX F.

REPORT OF COMMITTEE ON THE EFFECT OF THE EARTHQUAKE ON HIGHWAY STRUCTURES.

COMMITTEE:
MARSDEN MANSON, C. E. MOORE, E. F. HAAS.

In carrying out the matter of inquiry into damages to highway bridges, letters were written to all County Surveyors in the region affected by the earthquake. Only Alameda County has replied, so far, the reports being "no damage;" all bridges in the county are of non-reinforced concrete from 80 ft. (1) to 60 ft. (2), and then down to 3-ft. culverts. All large bridges have been carefully inspected—not a crack reported in any.

DISCUSSION.

Mr. Duryea.

EDWIN DURYEA, JR., M. AM. SOC. C. E. (by letter).—The writer wishes to call attention to one of the conclusions of the Committee on Fire and Earthquake Damage to Buildings which he believes to be too severe to be warranted by the facts. He recognizes that the members of this Committee had more opportunities to judge of the effects of the earthquake on buildings than he had, but, notwithstanding this, he has incidentally seen a sufficient number of instances of ordinary brick buildings which were not seriously damaged by the earthquake to make the very severe conclusions of the Committee as to such buildings seem to be unjustifiable.

The particular statements to which exception is taken are the following:

"The building with stone, brick or block construction, having horizontal mortar joints, does not answer the requirement (elasticity) at all. It may be stated, as one of the most obvious lessons of the earthquake, that brick walls, or walls of brick faced with stone, when without an interior frame of steel, are hopelessly inadequate. As a method of building in earthquake countries, such types are completely discredited."

And again:

"The writers simply reiterate the statement that, speaking generally, buildings of brick walls and wooden interiors cannot be built which will not be wrecked in a severe shock, it being a fault of design and not of materials or workmanship."

This conclusion is so severe that, if it were true, it would entirely exclude the safe use of ordinary brick buildings, for any purpose, on the San Francisco peninsula. It would seem that even the few instances of brick buildings which have withstood the earthquake with little or no damage would make such a conclusion unjustifiable. The writer has taken no pains to find such examples, but believes that he has seen a number of brick buildings which, while including no steel in their framework, have passed through the earthquake with very little damage. As specific instances, he would mention the brick church on the south side of Geary Street between Gough and Octavia Streets, the stone church on the southwest corner of Geary and Franklin Streets, and the brick cathedral on the northwest corner of Van Ness Avenue and O'Farrell Street. The cathedral does not appear to have been injured at all, the stone church only by the fall of its spire, and the brick church by only slight damage to its spire. The writer has no exact knowledge as to the framework construction of these churches, but, from their appearance, it is very improbable that any steel framework was used. It would seem that church buildings, with their open in-

teriors and large roof spans, would be especially subject to injury from earthquakes. Mr. Duryea.

On the morning after the earthquake, and before the fire had reached them, the writer twice passed by a number of brick warehouses on the north side of the Southern Pacific Railroad, between Fourth and Sixth Streets, and took a little time to examine them. They were of the commonest class of brick warehouse structures, neither design nor construction showing any special care, yet they had sustained relatively little damage; they also had the disadvantage of being built on extremely poor foundations, their settlement during the earthquake, with respect to the railroad right of way, just south of them, being about 2 ft. These warehouses were afterward entirely ruined by the fire.

It seems that, judged even by statements in the body of the report, the conclusion quoted is hardly warranted. In regard to damage to ordinary brick buildings, the Committee says:

"It must be recognized that buildings of this type were general throughout the region of the shock. The damage ranged from slight cracks to complete destruction of large structures, hence no general statement covers the case. A few individual statements may be of interest. In Oakland, a concrete-block building, 4 stories high, was injured little if any. At Palo Alto, two such structures, 2 stories high, probably not very well built, were completely wrecked, the blocks being broken and disintegrated."

After such a statement as this—that some brick buildings were but little damaged by the earthquake—and in view of the instances known specifically to the writer where ordinary brick buildings suffered but little damage, it seems that the very severe conclusion of the Committee is unwarranted; and that, recognizing the influence of good or bad foundations and of good or bad design, or material, some conclusion less unfavorable to this type of building would have been more just and more nearly in agreement with the facts. It is undeniable, of course, that such buildings are far from being as resistant against earthquake shocks as buildings of Type 3; the latter, however, are frequently entirely beyond reach because of their great cost; and the writer believes that the ordinary brick building, though generally inadvisable where financial consideration will permit of a better one, has shown, by numerous instances, that it may often pass through an earthquake practically unharmed, and, therefore, should not have such unqualified condemnation as that given by the Committee on Buildings.

The concrete pumping-pit of the Palo Alto municipal waterworks, about 50 ft. deep, was at first thought to be undamaged by the earthquake, as no cracks were visible in the walls. Later, however, it was found that where the well-casing entered the base of the pit, the joint had been broken by the earthquake so that when-

Mr. Duryea. ever the pump (a centrifugal pump placed on the bottom of the pit) was stopped, the pit rapidly filled with water nearly to the ground surface. Largely on account of the crowded condition of the pit and the interference of the work at the base of the pump, etc., the expenditure for repairs was about $1 300.

Mr. Riffle. FRANKLIN RIFFLE, M. AM. SOC. C. E. (by letter).—The writer fully agrees with the Committee on Fire and Earthquake Damage to Buildings that both wood and steel frame structures are admirably adapted to withstand earthquake shocks, but he is not prepared to endorse the unqualified condemnation of a type of construction which, after making due allowance for numerous instances of inadequate foundations and poor workmanship, has proven itself capable of much greater resistance to shocks than the Committee apparently is willing to concede.

The process of reasoning whereby the Committee arrived at its conclusions regarding brick buildings appears to be about as follows: An earthquake sets up, in a structure, a swaying motion, the tendency of which is to distort the frame; if sufficiently elastic, as is the case with wood, steel, and reinforced concrete construction, the structure returns to its original form after distortion; if not elastic, as in the case of stone, brick, or block construction having horizontal mortar joints, the structure is ruptured or wrecked. The latter type is then summarily dismissed with the phrases "completely discredited" and "hopelessly inadequate." It is true that the Committee cited a few examples of the failure of brick buildings, to give practical support to its theoretical deduction; but, leaving theory entirely out of the question, the citation of a few examples, coupled with the information that "the damage ranged from minor cracks in brickwork to the complete destruction of large buildings," does not seem to justify the following sweeping conclusions:

"* * * as one of the most obvious lessons of the earthquake, that bricks walls, or walls of brick faced with stone, when without an interior frame of steel, are hopelessly inadequate. As a method of building in earthquake countries, such types are completely discredited."

"* * * speaking generally, buildings of brick walls and wooden interiors cannot be built which will not be wrecked in a severe shock, it being a fault of design and not of materials or workmanship."

It is the opinion of the writer that the one great lesson taught by the earthquake is not the inadequacy of any one of the well-known types of building construction, but the necessity of adhering more rigidly, in all types, to the fundamental principles of good design, good materials and good workmanship. It is safe to say

that a careful investigation of all the building failures which resulted from the earthquake, would, in most cases, bring to light serious infractions of the commonly accepted rules for safe construction, which would account for much that the Committee would attribute to inadequate design or type. Mr. Riffle.

During the first day of the fire the writer spent several hours in the district in which brick buildings were the prevailing type, and made note of the fact that most of these were in good condition before the flames gutted their wooden interiors and reduced their walls to ruins.

If the Committee had included in its investigations the brick buildings in the Potrero and Mission Districts of San Francisco, which escaped the fire, it is doubtful if it would have condemned this type of construction in its otherwise excellent report. Almost without an exception, these buildings either were not damaged at all, or were damaged so slightly that the cost of repairs was trivial; and this applies to some buildings which had not been constructed in the substantial manner which engineers and architects would consider essential, even under usual conditions.

A brief description of a brick warehouse in the Potrero District, which came through the earthquake almost without a crack to bear witness to the severity of the shock, will show conclusively that it is not impossible or even difficult to construct a brick building with a wooden interior "which will not be wrecked in a severe shock."

The building is on the east side of Kansas Street, between 15th and 16th Streets, has a frontage of 100 ft., a depth of 200 ft., and is 4 stories high. The site is a portion of a tract of marshy tide land, which was formerly used as a dumping ground for the city's garbage. To secure a satisfactory foundation for the walls and columns, piles were driven and cut off below the water level, or about 10 ft. below the level of the street. The foundation walls were built of concrete reinforced by twisted steel bars, while, above the street level, the walls were constructed of common brick with cement mortar joints. The side walls contain no openings, but both the front and rear walls are generously provided with large windows on each of the four floors, in addition to six wide-arched doorways on the ground floor—three at each end.

The floor systems consist of wooden beams and joists supported by heavy wooden columns. A brick partition wall divides the building into two equal parts, each 100 ft. square. At the intersection of the partition wall with the north side wall, a portion of each of the two intersecting walls was carried to a height of about 20 ft. above the roof. These walls formed a support for a water tank about 15 ft. high and 12 ft. in diameter, which was full of

Mr. Riffle. water at the time of the earthquake. Both the building and the tank withstood the shock (or series of shocks) without sustaining any damage whatever. There was nothing unusual about the methods used by the architect. The work was done by contract under the personal supervision of an inspector acting under the architect's instructions. The results obtained naturally followed the careful observance of the commonly accepted standards of good building practice. During the past year the owner of this building has constructed two similar buildings in the same block. In the light of their experience, who will say that the owner and architect were not fully justified in adhering to a type of construction which the Committee has so mercilessly condemned as being "hopelessly inadequate"?

On Rhode Island Street, less than one block from the building already described, is a 2-story brick warehouse which was built, many years ago, on a foundation of solid rock. Although, for a period of several years, it has presented a somewhat dilapidated appearance from the fact that much of the mortar had disappeared from the joints, the walls were practically undisturbed by the earthquake.

On the opposite side of the street is a 5-story brick building occupied by the National Ice Company. Owing to the unusual height of the stories, the building is about as high as a 9 or 10-story modern office building, but it was so well constructed throughout that it suffered no damage whatever.

Two blocks farther north, on Eighth Street, is another 5-story brick building which was built many years ago, and, for a long time, was used as a sugar refinery; of late years, it has been used as a syrup factory. The floors and their supporting columns are of wood construction. At the time of the earthquake the top floor was heavily loaded with tanks of syrup. The building suffered practically no damage. This is certainly a remarkable showing, taking into consideration the character of the foundation, which is reliably reported to have sunk several feet during the erection of the superstructure. The walls were well tied together by iron rods, which largely compensated for the instability of the foundation.

Going west from the Potrero through the Mission District, one may see the following brick structures, the walls of which were uninjured: The Lick School building, and the Forderer cornice works, on 16th Street; the Bryant Street Power House, of the United Railroads, and its two tall chimneys; the Union Brewery and Malting Company's tall building, on 19th Street; the Mission High School building on 18th Street; the Spring Valley Water Company's pumping station and its chimney on 17th Street; the

tall brick chimney of the brick plant on West 16th Street; other structures of more or less importance could be included in this list. It is true that the number of brick buildings in this district forms an exceedingly small proportion of the total number of buildings—which are chiefly frame—but surely evidence of this character is entitled to the same consideration as that mentioned by the Committee in discussing the behavior of the limited number of steel frame buildings in the city, which were constructed with far greater care than the brick buildings just mentioned. Mr Riffle.

A brick building on Beale Street near Market, owned and occupied by a wholesale hardware company (Dunham, Carrigan and Hayden Company), was constructed upon what the architects termed a "floating" foundation, which consisted of an independent grillage of heavy timbers for each of the four walls. The floors were supported by cast-iron columns. With the exception of a few bricks which were dislodged by the shifting of the heavy cornice stones, the foundation and walls were uninjured by the earthquake, although the latter were reduced to ruins by the fire. The foundation is still intact, and will be used for another building.

The writer has not overlooked the fact that not a few brick buildings were partially—and some wholly—wrecked by the earthquake, but it is generally conceded that most of these were so flimsily constructed that their failure was plainly due to poor materials, poor workmanship, or a combination of the two.

The City Hall is a case in point. To many, its wreckage was no surprise, since it had long been regarded as a notorious example of vicious construction. The massive columns, which were a prominent feature of the building, consisted of cast-iron shells presumably filled with good concrete. The earthquake, however, developed the fact that loose sand, brickbats, and building refuse generally, had been largely substituted for concrete. The walls were constructed with an equal disregard for honesty and good practice.

In several other instances of partially wrecked brick buildings known to the writer, the cause of failure was clearly due to inadequate foundations. Two of these had been constructed on "made" ground, while another had been built partly in excavation and partly on embankment; in fact, poor foundations were responsible for many failures of buildings of this class. Had all the brick buildings of San Francisco been erected on as substantial foundations as those prepared for the steel frame buildings mentioned by the Committee, and had they been given equally rigid inspection during their construction, it is not too much to say that the percentage of failures would have been very small indeed. On the other hand, had the steel frame buildings been erected on inadequate foundations, there would have been a large percentage of failures recorded against this type of construction.

Mr. Riffle. The writer is decidedly of the opinion that a careful, unbiased inquiry into all the facts pertaining to the behavior of brick buildings at the time of the earthquake, together with the facts regarding their construction, would not only show that, to a great extent, the conclusions contained in the reports were based on insufficient evidence and erroneous deductions, but would lead to a modification, if not a complete reversal, of the Committee's findings.

Mr. Wadsworth. H. H. WADSWORTH, M. AM. SOC. C. E. (by letter).—Lest the report of the Committee on the Effect of the Earthquake on Highway Structures should convey the impression that such structures were singularly immune from damage by the earthquake which caused such destruction to other engineering structures in the vicinity of the region of greatest movement, the writer presents a photograph (Plate LIV) of a highway bridge across a creek tributary to Tomales Bay, near Point Reyes Station on the North Shore Railway, in Marin County, and within less than 2 miles of the fault line. This bridge was seen by the writer about three months after the earthquake, and had then been made passable for teams.

The bridge, originally, had eight panels, its total length being approximately 120 ft. It is located in a direction nearly north and south. The abutments are of piles and timber. The effect of the earthquake was to settle the north abutment some 2 or 3 ft., and move it southward so much that, in patching up the structure temporarily, the north end panel was not utilized as a part of the span, but the second batter post of each truss was given a footing on the abutment in its new position, thus becoming the end posts of the trusses. An apron was then built from the approach to the bridge roadway.

The stresses in the structure were thus considerably altered from those originally existing, but the bridge was still doing duty, and apparently no restrictions as to its use had been imposed by the county authorities.

Mr. Perrot. EMILE G. PERROT, ASSOC. M. AM. SOC. C. E. (by letter).—The report of the Committee on Fire and Earthquake Damage to Buildings, while very comprehensive, is lacking in some details as to the exact nature of the materials used in construction, a knowledge of which would throw more light on the subject. Some of these details, which the writer feels would be of much benefit to those interested, are the following:

Under the head of masonry walls: What was the character of the brick; how were the walls bonded; what was the composition of the mortar (lime or cement); and what was the thickness of the walls, compared with the standards of the New York or Philadelphia Building Bureaus?

HIGHWAY BRIDGE NEAR POINT REYES, MARIN COUNTY, CAL.

Under the head of reinforced concrete: What proportions were used in the concrete; what kind of stone was used in the aggregate; and how many inches of concrete protected the rods on the bottom and sides of beams, girders, and slabs? Mr. Perrot.

All these facts are of vital importance when comparing results. In most cases it is not possible to determine these details from an inspection of the work, but it might be possible to obtain records, such as specifications of the work (if they have not been burnt), which would give this information.

The Committee states that the quality of the workmanship had an important bearing on the results; that at Stanford University, where brick walls were veneered with stone, "there was a separation of the ashlar facing from the brick backing," while the buildings with full stone walls were but slightly injured. This would seem to indicate a much lower standard of workmanship in the former than in the latter case, for it is certainly not a difficult matter to bond an ashlar face to a brick backing. Further, it would lead one to believe that the standard of workmanship on the Pacific Coast was not as high as in the East, especially as the buildings at Stanford University were supposed to be of the best; what then could be expected of those buildings for which the cost had to be more carefully considered?

The question is sometimes asked: What would happen to New York, if an earthquake occurred there? The writer does not think that a calamity, such as befell San Francisco, could befall any of the large Eastern cities, as the buildings are much better constructed in every respect.

Under the head of fire-proofing, the Committee considers solid concrete—it does not say in what proportions—at least 4 in. thick, with a mesh of reinforcement, equal to red brick set in Portland-cement mortar. It also states "that all structural parts of a building, of whatever material constructed, must be protected by another material which will be a more or less complete loss in fire," and refers to reinforced concrete as coming under this head. The writer cannot see the force of this argument. Since it is agreed that reinforced concrete is a good fire-proofing material, why not make the integral structural part larger in the first place, and do away with the veneer or covering, as it adds nothing to the strength of the member, while an additional thickness of concrete, being homogeneous with the structural part, gives it increased strength; so that, prior to any fire, the building has a greater factor of safety, and, in the event of a fire, the portions of the exterior surfaces of the concrete rendered worthless can be removed, and the parts resurfaced? The writer has seen this done with concrete damaged by frost to a depth of from 2 to 4 in.

Mr. Perrot. To illustrate the point: If it were calculated that a 10 by 10-in. reinforced concrete column would be required to support a certain floor load, and if 3 in. of concrete were added all around for fire-proofing, making a 16 by 16-in. column, this column would not occupy any more floor space than a 10 by 10-in. reinforced concrete column, fire-proofed with 3 in. of metal lath and plaster or terra cotta, while there would be two and one-half times more concrete, and, consequently, two and one-half times more strength in the former column than in the core of the latter, as the plaster or terra-cotta veneer would add very little to the strength.

With regard to reinforced concrete buildings offering the solution to the problem of resisting earthquake shocks, the Committee seems to think that, as now designed, they are weak; this may be so, but it must not be forgotten that, in the East, very many large and important buildings have been built, and are being built, in which much thought has been given to just such points as those to which the Committee would call attention.

Mr. Wagoner. LUTHER WAGONER, M. AM. SOC. C. E. (by letter).—The breaks in tall chimneys, as far as observed by the writer, were in the middle third; the smaller chimneys, upon residences, were usually broken at the roof line, many being twisted and broken through a number of courses. One chimney, upon a brick wall, 18 by 32 in., and 7 ft. above the wall, was broken through eight horizontal courses, its top being rotated about 30° from its original position. From a careful inspection, both before and after the fire, the writer believes that most of the failures were due to lack of bond, and that the addition of a very moderate compression by the use of one or more pairs of rods, reaching preferably from base to top, and provided with turn-buckles to maintain tension upon the rods, would reduce chimney accidents to a very low percentage. The same remarks apply to walls, gables and cornices.

The writer concurs with the conclusion reached by the Committee on Fire and Earthquake Damage to Buildings that, "the prime requisite of the structure is elasticity," and, for this reason, advocates the use of moderate compression by long rods, where possible, as a proper precaution in earthquake countries. Such rods could be adjusted to maintain an elastic compression upon the top courses, and would aid in checking, and possibly wholly prevent, rotation as well as overturning.

Concerning the Committee's conclusions as to fire damages, the writer draws different ones from the same premises. He watched the burning of many of the buildings supposed to be fire-proof, and believes it is safe to say that little damage was done to the external trim by fires from adjoining buildings, but that the greater portion of such damage was due to flames issuing from the window

and door openings of such fire-proof buildings. That this is a proper conclusion is evident from the fact that the greatest damage was at such openings, and that practically no damage occurred between the openings. Mr. Wagoner.

The only logical inference that can be drawn from the carefully compiled data of the Committee, as far as concerns the spread of a fire or the arrest of its progress during a general conflagration, is that a solid fire-wall is required to check the flame.

All these high-class buildings had large window areas—common glass in wooden sashes and frames—wooden floors and doors, and much wooden furniture, and had no more resistance to exterior fire than ordinary brick or wooden buildings.

The chief lesson to be learned from this is that such openings should have better protection.

The report of the Special Committee of the National Fire Protection Association, on the Baltimore fire, 1904, page 123, states:

"The general absence of protection at exposed wall openings is responsible for the spread of this conflagration more than any other cause. In fact, this condition may be safely stated to have been the cause for the spread of this fire beyond Fire Department control."

Again, page 124:

"The contents of a fire-resistive building, without proper subdivision and no adequate protection against exposing fires, are scarcely any safer as regards destruction by fire than if contained in a building of ordinary construction."

So true and generally applicable are the foregoing quotations that they deserve to be framed and hung in the office of owner, designer and architect. Indeed all the conclusions of the above-named Committee on the Baltimore fire, pages 118 to 124, could also be deduced from the San Francisco fire.

It was stated at a meeting of the Structural Association of San Francisco that any building had about 60% of chance of fire from an exterior attack. This was said to be due to the fact that it usually had the four sides and top as exposures, as against an interior attack; but, in a general conflagration, the attack is entirely an exterior one, hence the greater necessity of proper guards for all exterior openings and roofs.

EUGENE W. STERN, M. AM. SOC. C. E.—The information gathered by the various committees is of the greatest value, especially because, for the first time in history, the effects of a very severe earthquake on tall, modern, steel-frame buildings have been made manifest. Mr. Stern.

Moreover, in view of the recent disastrous experiences at Valparaiso and Kingston, with the attendant loss of life and property, the subject is a timely one, and the thanks of the profession are due to

Mr. Stern. the gentlemen who have so carefully and thoroughly studied the subject at first hand, and given the results of their investigations in these reports.

The Committee on Geology makes some statements that do not seem to be entirely in accord with the conclusions of the Committee on Buildings. It states:

"The damage to buildings, while large, was confined to those faulty either in design or in construction."

And:

"The results, so disastrous to buildings and engineering structures, can and will be provided against in the future."

This statement is not concurred in by the Committee on Buildings, which, in its report, says:

"Obviously, the shock may range from a tremor to that of a violence that would wreck any building."

And also:

"Should the earth-slip take place beneath a building, it would be wrecked."

To prove that the quality of workmanship, while important, is still not sufficient to prevent a disastrous wreck, the Committee on Buildings cites the case of St. Patrick's Academy, in which the brick walls were so well laid up in Portland-cement mortar that the walls invariably broke through the brick instead of the mortar joints.

The first thought that comes to one's mind is that a limit should be placed on the height of buildings. The higher the building, the more disastrous must be the whipping and gyratory effect of the earthquake. It is to be regretted that the Committee on Buildings makes no recommendation on this point.

It reports that while the injury to steel frames in buildings of Type 4 (in which the entire structure, including brick walls, was carried on the steel frame) was inconsiderable, the enclosing walls of brick or mortar were damaged to a greater or less extent, and will require extensive repairs. It cites the case of the Claus Spreckels Building, in which the stonework was badly worked on the beds above the tenth story; also that of the Chronicle Building and others in which the brick, stone, and terra-cotta work were badly cracked. While it recommends that for rear walls reinforced concrete offers the best solution, the reinforcing members being tied to the steel frame, it is illogical in recommending that the fronts be made of brick, terra cotta or stone tied to the steel frame, in so far as its own report shows that the motion of the earth tended to shatter the bond in brick and mason work.

If brick and masonry walls shake loose during an earthquake shock, let these materials be abandoned for something better.

Mr. Stern.

It would be of great interest to know what damage resulted to buildings of Type 4, which were located on or near the fault line. The reports have not mentioned any cases of this kind.

As a class, the steel frame seems to have been the only form of construction that came through the earthquake at all satisfactorily, although the Committee reports damage to structural frames that will have to be repaired.

It states that the prime requisite of the structure is elasticity, which is reasonable; but should not provision also be made for inequality of settlement of foundations, as well as the vibratory effect of the quake? It is not always certain, in earthquake shocks, that the soil under foundations will not settle.

The speaker cannot conceive that a thoroughly rigid structure is the right one to build, but rather one that is designed for vibratory and swaying motion, and, at the same time, has sufficient flexibility in the joints, so that in case of unequal settlement of foundations it can be repaired at very much less expense than one in which no allowance for this has been made.

Regarding foundations, the Committee states that no instances of damage have come to its notice. Does this mean that there has been no unequal settlement of foundations, or that none of the foundation piers have been shattered, or what is the meaning? The speaker would recommend that isolated piers be firmly tied together at the level of the bottoms of columns.

The catastrophes at San Francisco and Kingston are still fresh in our minds, but there have been other very destructive earthquakes in comparatively recent times, and it might be well to review briefly some of the most disastrous of these.

In 1822, in Chili, an earthquake shook the coast for a distance of 1 200 miles north and south, and the coast line was elevated from 4 to 5 ft. for a distance of 35 miles.

In 1835 and 1837, also in Chili, there were disastrous earthquakes in which towns were destroyed and the coast was raised to a height of from 4 to 8 ft. for a distance of more than 100 miles.

In 1891, in the Provinces of Mino and Owari, in Japan, a severe earthquake occurred in which 7 000 people lost their lives and 17 000 were wounded, 197 000 houses were wholly demolished and 78 000 more than half demolished. A fault occurred, which extended entirely across the island, the vertical displacement being from 9 to 18 ft. and the horizontal displacement about 6 ft. A levee was moved bodily 60 ft., carrying a thicket of bamboo and pine trees. The area shaken was estimated at approximately 95 000 sq. miles.

In 1897, in the Provinces of Bengal and Assam, in India, there was an earthquake the effects of which extended over 1 750 000 sq. miles, many dislocations of the earth taking place, both vertical and horizontal, amounting in some places to 30 ft. The violence was so

Mr. Stern. excessive that not a masonry structure in a district covering 6 000 sq. miles was left standing. Monuments and tombstones were destroyed, and trees 6 and 7 in. in diameter were broken through the trunks by the swaying motion of the earth.

The recent earthquake at Valparaiso, in August, 1906, caused a loss of more than $250 000 000 in property, and destroyed, besides, 140 smaller towns and villages. More than 1 000 people lost their lives.

In January, 1907, the City of Kingston, Jamaica, was destroyed, the loss of life being about 1 000.

The Province of Guerrero, Mexico, in April, 1907, was visited by an earthquake which destroyed several towns and caused a loss of life of more than 100.

M. de Montessus de Ballore has tabulated the number of seismic occurrences which have been observed during the nineteenth century, amounting to the enormous total of 131 292 quakes, and 10 500 epicentra.

In view of the foregoing facts, it must be admitted by the candid student that the problem of constructing buildings in earthquake districts so as to withstand entirely the violence of some of the excessive seismic disturbances is a difficult one.

Destruction of property cannot be prevented, but the excessive loss of life and property through the improper construction of buildings can and should be very largely reduced.

It is an engineering possibility to design a structure—a ship, for instance—that will stand any kind of strain, and adapt itself to all manner of conditions. It is also an engineering possibility to design a building so that it will stand a great deal of shock, besides some inequality of settlement of foundations due to a distortion of the earth supporting it.

A proper adaptation of such structures to the needs of those occupying them is a modifying condition which, together with the architectural requirements, has to be considered in the problem. Primarily, it is a question of applying sound engineering principles to the question in hand, and architectural features should be subordinated to these.

The important lessons of the San Francisco disaster, as regards the construction of buildings in earthquake districts, seem to the speaker to be:

1.—That buildings should not be constructed more than 100 ft. in height.

2.—That all structures in the business sections of the city should be fire-proof.

3.—That fire-proof structures should have a frame in the design of which swaying and gyratory motions, as well as unequal settle-

ment of foundations, should be provided for; and that this frame should carry the walls at every floor. Mr. Stern

4.—That the best material thus far known for such a frame is steel.

5.—That the floors, walls, and partitions should be of reinforced concrete in which the reinforcing members are securely tied to the steel frame.

6.—That the water and gas pipes, and electric wires, should have special ducts, and the practice of carrying these next to the columns and surrounded by their fire-proof covering cannot be too strongly condemned. There should be expansion joints in the water and gas pipes, in order to avoid breakage in case of settlement of the building.

J. K. FREITAG, ASSOC. M. AM. SOC. C. E.—Referring to the "Report of the Committee on Fire and Earthquake Damage to Buildings," especially that portion dealing with the fire damage, it would seem that certain broad and vital principles relating to fire-resistive construction in general have not been sufficiently emphasized, but have been made of secondary importance to descriptions of damage of mere structural detail, and to consequent recommendations deduced therefrom. Mr. Freitag.

The first conclusion* of the Committee is:

"Any deductions from the fire must be those based upon a general conflagration, and not those of an isolated fire. In view of the complete destruction of all materials, it becomes a question as to what should be done to make a building fire-proof."

This conclusion, although somewhat amplified in a succeeding paragraph, which will be discussed later, seems to place altogether too much emphasis upon the possibility of making an individual structure fire-resisting under conflagration conditions. Previous conflagrations, notably that of Baltimore, have shown the almost utter futility of attempting to cope with fire when once it has reached the proportions of what is generally called a conflagration. No structural materials of which we have present knowledge are equal to the task of resisting successfully such severe test conditions, at least to a point which would justify any reliance which might be placed upon their use.

Further, if the high temperatures, the multitudinous points of attack, and the general impotence of all fire-fighting measures obtainable during a conflagration are to be entirely overcome, then the task of fire-resistive construction is surely a hopeless one, and those interested in fire-resistive principles might well be dismayed at the records of Baltimore and San Francisco. The Committee evidently realized this aspect of the question, for it states that:†

"Unless one has been an eye-witness, it is difficult to realize how all materials that men make into the shape of buildings can be so utterly destroyed in a general conflagration."

* Page 241.
† Page 237.

Mr. Freitag. If, then, that fact is once admitted, as it must surely be by those thoroughly familiar with the tests of fire-resistive methods afforded by the Baltimore and San Francisco conflagrations, the main question becomes, not so much what can be done to make any particular building fire-proof, using this or that detail of concrete or terra cotta, or one or another construction of exterior walls or partitions, etc. (though all these points are of immense value as details of the larger problem), but what can and should be done to prevent conflagrations? The Committee's answer to this most vital problem of the American people is far from satisfactory.

Statistics as to the annual fire losses in the United States are probably familiar to all who are interested in building construction. Let it suffice here to recall a few figures. The fire loss in the United States has now approximated $200 000 000 annually for several years, during normal conditions, *i. e.*, years which have not included widespread conflagrations. Add to this direct loss the expense of maintaining fire departments, etc., and the money paid to fire insurance companies in the way of premiums, and it is questionable whether these financial losses do not equal, or even surpass, the value of all new buildings erected throughout the United States within the year for which the comparison is made. Including the San Francisco conflagration loss, the year 1906 becomes an abnormal one, and the fire-waste alone, not counting fire departments or insurance premiums, is estimated at not less than about $506 000 000, while the estimated value of new building operations in the United States for the same year probably approximates $575 000 000. Hence no more vital economic problem is before our people to-day than the lessening of this stupendous drain.

Before considering the responsibility which lay before the community of San Francisco in making impossible a repetition of at least their share of this fire loss, or the same responsibility which lies before all cities, it will be pertinent to inquire into the conditions which made this great calamity (by fire) possible.

The report of the Committee states that:

"San Francisco was built probably in about the same way as other cities. It is an error to say that it was a wooden-frame city, as the business district was generally composed of buildings with brick walls. In among these had been constructed the so-called fire-proof structures, exposed on all sides to danger by the burning of the inflammable structures around them."

That San Francisco was not considered by insurance interests to be built "in about the same way as other cities," is made most clear in the Report on the City of San Francisco, issued in October, 1905, by the "Committee of Twenty" of the National Board of Fire Underwriters. The general summary of that report, page 64, reads as follows:

"CONFLAGRATION HAZARD.—*Potential Hazard.*—In view of the exceptionally large areas, great heights, numerous unprotected openings, general absence of fire-breaks or stops, highly combustible nature of the buildings, many of which have sheathed walls and ceilings, frequency of light wells and the presence of interspersed frame buildings, the potential hazard is very severe. *Probability Feature.*—The above features combined with the almost total lack of sprinklers and absence of modern protective devices generally, numerous and mutually aggravating conflagration breeders, high winds, and comparatively narrow streets, make the probability feature alarmingly severe. Mr. Freitag.

"SUMMARY.—While two of the five sections into which the congested value district is divided involve only a mild conflagration hazard within their own limits, they are badly exposed by the others in which all elements of the conflagration hazard are present to a marked degree. Not only is the hazard extreme within the congested value district, but it is augmented by the presence of a compact surrounding great-height, large-area frame residence district, itself unmanageable from a fire-fighting standpoint by reason of adverse conditions introduced by the topography. In fact, San Francisco has violated all underwriting traditions and precedent by not burning up. That it has not done so is largely due to the vigilance of the fire department, which cannot be relied upon indefinitely to stave off the inevitable."

The last sentence of this summary has proved only too true a prophecy of the calamity which followed only a few months after the publication of this report.

In discussing particularly the congested-value district, which contained about 2 100 buildings, *Insurance Engineering* gave the following summary of the types of building construction:

Fire-proof	2.2 per cent.
Brick and wood joist	68.3 " "
Frame construction	29.5 " "

In further description of this area, the same journal stated as follows:

"In the congested value district there was but one sprinkler equipment and it was practically obsolete. One hundred and twenty buildings contained stand-pipes and hose, and a few small fire pumps drawing from deep wells. * * * All the fire-proof buildings had windows unprotected, and in many cases were badly exposed. About 50 per cent. of the joisted brick buildings had furred and wood-sheathed walls and wood-sheathed ceilings. * * * Large, open light wells were numerous, and conspicuously prominent on account of their size and number. Several of the older hotels were literally perforated with them. * * * About 90 per cent. of the entire city and 30 per cent. of the business section were of frame construction, brick buildings being confined almost entirely to the fire limits, and San Francisco could be justly termed a wooden city."

Mr. Freitag. The above criticisms of San Francisco's buildings would be severe enough—indeed far too severe to be pleasant, as applied to any large city—but, when applied to a city in a locality subject to seismic disturbances, the facts become a severe indictment, and show either the indifference of San Francisco's citizens, or the ignorance or carelessness of the Building Department. Municipal regulations (whether or not in a locality subject to earthquakes) which permit 30% frame construction within a congested-value district, which permit buildings of masonry walls and timber floors and partitions to be eight stories, or 100 ft., high, which permit "large private hotels and apartment houses of this type," which provide no limitations as to undivided areas, and which allow the "absence of modern protective devices generally," are most certainly insufficient, short-sighted, and unjust, and most particularly unjust to those investors who improve their property by erecting steel-skeleton fire-and-earthquake-resisting buildings, and then have them "exposed on all sides to danger by the burning of the inflammable structures around them."

This brings up the consideration of civic responsibility, as applied to building construction. European cities long ago learned the lesson that safety to the individual means safety to the whole community, and *vice versa*. Witness the most ineffectual fire departments in most continental cities, and, withal, the trifling fire-losses, and, especially, the almost total absence of conflagrations, or even the spread of fire to immediately adjoining property in the cities of civilized Europe. The reason is to be found entirely in the matter of building construction. A building which will prove a menace to neighbors cannot be erected, and the responsibility of the individual as affecting the community is even carried so far in some localities of Europe that the owner of property causing fire damage to neighbors is held financially responsible for such loss.

It is just some such civic responsibility which is needed, and needed very soon, in all large American cities. Responsibility of the individual to the community, which will cause the individual to contribute to the public safety in matters of building construction by erecting structures which will not prove a menace to his neighbors; and responsibility of the community to the individual, in that those investors who improve their land by the erection of costly and permanent structures shall not be allowed to suffer constant hazard through irresponsible neighbors who have no thought or care of their civic duties.

It is now a trite saying that "fire-proof buildings must stand in fire-proof cities," but this statement contains the whole truth of the matter of fire-resistance. If American cities are not to suffer such conflagrations as have occurred at Chicago, Boston, Paterson, Baltimore, and San Francisco, besides many other lesser ones; if the reali-

zation of this tremendous financial drain is once grasped in an effort to lessen it; if it be admitted that isolated buildings surrounded by severe risks cannot withstand conflagration conditions, then the achievement of fire-proof cities (or at least the congested areas therein) must be made possible by uniform fire-resisting construction throughout. Mr. Freitag.

Touching upon this point the Committee states as follows:

"The only statement that can be offered is that the best insurance for buildings would be the isolation of a district containing nothing but fire-proof structures. A general conflagration would then be impossible. Manifestly, this is impossible in San Francisco, where business must be resumed with the least cost. In many cities, it would be good insurance for men owning large buildings to combine to buy out old and inflammable structures, either demolishing or rebuilding them. Otherwise, there remains the danger of general conflagrations, such as those at Baltimore and San Francisco, in which fire-proof buildings will be injured from 30 to 60 per cent."

With the first statement, regarding the isolation of congested districts to contain nothing but fire-proof structures, the speaker most heartily concurs, not only because such practice would afford the "best insurance for buildings," but also because it would afford a maximum of safety and assurance for the whole community. With the statement, however, that "this is impossible in San Francisco, where business must be resumed with the least cost," or, indeed, that uniform fire-resisting construction is impossible in any city ready to take the lead; and also with the suggestion that "it would be good insurance for men owning large buildings to combine to buy out old and inflammable structures," etc., instead of throwing some such responsibility upon the city itself, the speaker begs to take most decided exception.

In the United States we are so prone to consider the rights of the individual that we are apt to overlook the rights of the aggregation of individuals. It is not denied that municipal building regulations adopted by any city, requiring uniform fire-resistive building construction after any fixed date, would give rise to seeming injustices and hardships, but if laws requiring the remodeling of present risks were also rigidly enforced, in addition to laws covering the erection of new buildings, the hardships would soon be equalized, and benefit accrue to the community in the way of reduced fire losses, reduced insurance premiums, reduced expenses for maintaining fire-fighting equipments, and added security to life and property interests.

"The fire problem, meanwhile, is with us, and it is not only for future buildings that we are concerned but with those that exist at present. Out of 300 000 buildings in the Metropolitan district (New York) not more than 2 000 are classed as thoroughly

Mr. Freitag. incombustible. With the remaining 298 000 we have got to deal. That these buildings can be made practically fire-proof—the word is used advisedly—is a statement which is not only capable of demonstration, but has already been demonstrated by fire experience.

"To safeguard these buildings is an engineering proposition pure and simple. The knowledge and ability are available and there is required simply a desire on the part of occupants and property owners. A very large percentage of the causes of fires is known, and can be eliminated from a building; there are on the market devices and systems of proved worth for detecting such fires as do occur, and for extinguishing them by automatic or manual means; a building can be planned to limit the area of fire; it can be prepared to facilitate the work of the public fire department; it can be prepared to reduce the loss resulting from fire and water and thereby reduce the fire tax. This outlined plan makes a standard for judging fire conditions, and if applied to any particular building, it would register in a definite and precise form the degree of consideration which the owner or occupant has given to the danger of fire."*

Mr. Harts. W. W. HARTS, M. AM. SOC. C. E. (by letter).—To those who were present during the recent San Francisco earthquake, it requires no stretch of the imagination to believe that there might easily occur an earthquake of such intensity as to defy human skill to construct an edifice, anything like those now built, which would be able to withstand its power. The writer's experience in Manila in December, 1902, led him to think, at the time of the San Francisco earthquake, that the results in the city would not be very serious. That earthquake was not only longer in duration than the San Francisco shock, but was believed to be greater in the amplitude of the vibrations. The damage in the Philippines was slight, as buildings are purposely kept low, and are erected cheaply, with the earthquake hazard constantly in mind. Had it not been for the destructive fire in San Francisco, it seems safe to say that the earthquake and its results would have been only a nine days' wonder.

To what extent it seems necessary to provide for earthquake stresses in buildings, is a question of great nicety and careful judgment. It can scarcely be stated in pounds per square foot of superficial area, like wind pressure, which soon reaches a limit, although this method has been suggested. It partakes far more of the nature of insurance, so that, although there might be placed no limit to the expenditure for the protection of buildings from damage from this source, still an economical consideration will very quickly place a limit beyond which it is not desirable to provide against such risks.

* *The Journal of Fire.*

Mr. Harts.

The probability of the recurrence of dangerous shocks, and the estimate of the intensity can only be surmised. The additional expense, required in a structure for protection, therefore, would appear rather to be some percentage of the value of the building, if such protection were to be calculated upon a proper basis. That some protection for valuable buildings is advisable, is borne out by the admirable resisting powers of the Call Building. On the other hand, it is easily conceivable that small buildings might be built satisfactorily without any special reinforcement, should it be more desirable to run the risk of destruction than provide for protection. It is unquestionable that the class of buildings resisting the earthquake stresses most satisfactorily have been those in which a certain amount of elasticity was found, permitting the building to yield to the shock and return to its true position without injury. There appear to have been many cases where brick buildings, well constructed, complied with these conditions, and the many examples of brick buildings standing after both the earthquake and fire had done their worst, is a sufficient indication that the sweeping assertion cannot be made as to the "hopeless inadequacy" of this type of structure. It is very dangerous, in any scientific discussion, to formulate sweeping general rules, and, in view of the evidence, the statement of the Committee, with reference to brick buildings, should be more explicitly modified. Furthermore, the explanation has been made that the test of engineering structures is not experience, and that the majorities cannot be relied upon, as in town meetings, to discover the truth as to their stability. Can it be maintained that the science of engineering is not the resultant of years of experience? Can it be said that successes are not guides and that failures are not warnings? The fact that wooden composition bridges for railways yielded to steel arose through no fault of the composite type, but through other causes, such as the increase in weight of rolling stock, increase in cost of wood and its short life, but mainly through the decrease in the cost of steel. Such a change has unquestionably been based upon economical and not altogether on theoretical reasons.

One of the earthquake features which does not seem to have been mentioned is the rotary effect of the shock. Evidence of this came to the writer's notice in examining the different gas holders on the North Beach water front immediately after the earthquake. In all the cases examined these holders were twisted in the same direction —counter clock-wise. Each one of the holders was twisted about 2½ ft., breaking the rollers from the guides. Of course, the holder was useless until put back into its place and readjusted. The actual motion of the earth was very slight, but the effect on these structures was very curious and noteworthy.

Mr. Rix

E. A. Rix, M. Am. Soc. C. E. (by letter).—The writer has often thought that the geographical structure of the San Francisco Penin-

Mr. Rix. sula has something to do with the character of the vibration which results at different points. For example: The writer's residence is on the summit of Russian Hill, on the solid rock, where the strata appear to be very much uptilted. The shock there seemed to be felt principally in a vertical direction, and the writer was not conscious of any great horizontal movement. The shock on the hill was severe, but in his rooms nothing was thrown from the walls. In one room there was a pedestal, about 4 ft. high, on which was a statue which had an insecure base, but this was not disturbed. The writer, therefore, has wondered whether the vibration was not reflected by the underlying stratum; or whether the severe horizontal movement, felt in the lower parts of the city, was not translated, or a portion of it, into vertical movement when it reached uptilted strata.

Mr. Whitaker. W. F. WHITAKER, JUN. AM. SOC. C. E. (by letter).—On the day of the earthquake the writer was running some levels in Bakersfield, Cal., and had great trouble with the bubble of his instrument. He knew that there had been an earthquake, but did not know that the quakes were continuing that afternoon. After the instrument had been nicely leveled, the bubble would move to one end of the tube. At first it was thought that the trouble was with the instrument, and another attempt was made to level it. After one or two trials of this kind it was discovered that if the instrument were left alone the bubble would move back to the center or perhaps to the other end of the tube. During the entire afternoon the same trouble occurred at intervals. The shocks were too slight to be felt.

The shock in the morning was felt by many, and caused some swinging of hanging objects. Although the writer was awake at the time, he did not know there had been an earthquake until told of it by others. In the evening, in discussing the peculiar action of the level bubble during the afternoon, the writer expressed the opinion that the shocks must be continuing, and learned later that this was true.

Mr. A. M. Bienenfeld. A. M. BIENENFELD, M. AM. SOC. C. E. (by letter).—A building under construction in Palo Alto, coming within the writer's notice, in which lime mortar was used, was badly wrecked, and it would seem that its destruction might be attributed to the use of the lime mortar. If all cement mortar, and no lime mortar, had been used, there would have been fewer wrecks of brick buildings.

The earthquake was not felt appreciably in Bakersfield, yet all the 70 or 80 steel tanks of the Standard Oil Company in that locality, which were filled with heavy viscous oil, were affected more or less. The tanks show that the oil spilled to the southwest, indicating that there was some tipping motion, or some motion of translation, sufficient to throw the oil in that direction. If there had been a return motion—oscillating, or vibratory—the oil would

have spilled to the northeast, also, but the latter effect was not noted. It took about 2 hours for this phase of the earthquake shock to manifest itself at Bakersfield. Mr. A. M. Bienenfeld.

ARTHUR L. ADAMS, M. AM. SOC. C. E. (by letter).—It seems to be a question of definition and degree. What constitutes a severe earthquake, as the Committee uses that term, and of what degree of severity does it consider the earthquake of April 18th, 1906? Granting that earthquakes may be so severe as to wreck any class of structures, what degree of severity is to be provided for in the design of buildings? Mr. Adams.

It would seem that the Committee must define its position on these points in order to be fully understood.

S. G. HINDES, M. AM. SOC. C. E. (by letter).—At the time of the earthquake the writer lived in an old house on Russian Hill, at the corner of Green and Leavenworth Streets. The house was built in 1860, and withstood the earthquakes of 1862 and 1868 without damage. The walls are entirely of concrete, 12 in. in thickness throughout their entire height, and rest directly on the solid rock. Four years ago, in making two large openings through the original walls, one laborer was able to cut a doorway in about two days, thus showing that the concrete had very little strength, there being probably very little cement used in it. Although the earthquake was not as severe on Russian Hill as in lower parts of the city, it was severe enough to unhook pictures and throw them across the room, but these walls were practically uninjured. The inside plaster was somewhat cracked, but the walls themselves are apparently unhurt. There is no reinforcement whatever in the walls. Mr. Hindes.

LOREN E. HUNT, ASSOC. M. AM. SOC. C. E. (by letter).—The writer desires to call attention to a few notable examples of brick construction which successfully withstood the earthquake shocks. All San Francisco engineers are familiar with the buildings and will agree that they represent good brick construction. They are the older Federal buildings in San Francisco. The "Appraisers Building" suffered practically no damage. The "Mint" was less damaged than the modern steel Post Office two blocks away. Most remarkable, however, is the case of the old Government building now known as the "Sailors Home," which was condemned as "unsafe" thirty years ago, and yet came through the disaster practically unharmed. It is not affirmed that good construction is the only reason that these brick buildings are still standing, but good construction was certainly an important factor. Their preservation is a tribute to the care and efficiency of the Government engineers. Mr. Hunt.

BERNARD BIENENFELD, M. AM. SOC. C. E. (by letter).—In the earthquake countries of the Far East, it is the practice, as elsewhere, when putting up substantial buildings, to go down to solid Mr. B. Bienenfeld.

Mr. B. Bienenfeld.

material, preferably rock, for foundations. The overlying loose material, or earth, is thereafter cut away and kept clear from contact with the foundation walls. It is the excessive motion and jelly-like tremors in the loose earth that cause the greatest damage in an earthquake shock. By thus avoiding contact between the comparatively incoherent superficial earth and the walls of the building, the vibration is minimized. It would be well to follow this practice in San Francisco. This might necessitate the use of curb or slight retaining walls to confine the loose earth; and such walls, both in their footings and on their faces, should be separated from the main building walls. Such a construction would add somewhat to the expense of construction; but the additional first cost would be a very cheap and efficient insurance against future disaster.

Regarding the contention of the Committee that the ordinary brick construction is inelastic, observation of the action of the brick walls of wrecked buildings that were being torn down subsequent to the earthquake and fire would indicate that there is indeed a considerable and surprising internal elasticity in this form of construction. Wherever good work had been done, and where the foundation was firm, brick buildings passed the ordeal of the shock remarkably well.

It is undoubtedly true that a proper reinforced-concrete construction on a proper foundation is among the forms best calculated to resist earthquake shocks; but it is none the less evident that a properly founded and constructed brick building has earthquake-resisting features considerably greater than the Committee has indicated.

It is to be deeply regretted, however, that the class of brickwork now being done in the rehabilitation of the city is generally of a nature to invite serious disaster in future severe earthquakes. The throttling restrictions of the labor unions, and the indifference and incompetence of the municipal building authorities are particularly blameworthy.

Mr. Carlin.

J. P. CARLIN, ASSOC. M. AM. SOC. C. E. (by letter).—Concerning the reviews and discussions of the conclusions reached by the Committees of the San Francisco Association, the writer is glad to note the unanimity of opinion, regarding the criticism of the Committees' finding, with special reference to brick construction on the San Francisco Peninsula. The following conclusion was reached by the Committee:

"It may be stated, as one of the most obvious lessons of the earthquake, that brick walls, or walls of brick faced with stone, when without an interior frame of steel, are hopelessly inadequate."

This was not at all obvious to the writer, when he examined the ruins, and, he believes, is not the fact.

Mr. Carlin.

In determining the damage to any of the wrecked buildings, careful investigation was required in order to ascertain what proportion was due to the fire and what to the earthquake. The civic pride of the people of the ill-fated city prompted them to attribute nearly all the destruction to the fire, and consequently their opinion must be discounted for bias. On the other hand, it would seem that many visiting committees have reported upon the ultimate damage as they found it, and have failed to consider properly the earthquake damage independently of the fire.

The vigilance committee in San Francisco, or "Committee of Safety," organized immediately after the fire, used heroic measures in leveling the ruins; and, naturally, the buildings to suffer most were those since brought under the ban of the beforementioned Committee. Dynamite was used unsparingly, especially in razing unsupported brick walls, wherein fires had completely gutted the interior because they were not of steel frame or so-called fire-proof construction. Consequently, there are, or were, but few cases in which buildings of brick exterior walls were left standing; but, such as did remain are classic examples of slow-burning construction in championing the cause of brick buildings in San Francisco when properly built.

The writer made a personal examination and report on the ruins of the Palace Hotel, and with special reference to reconstructing it without razing the existing walls. This structure covered an entire block, in area about 250 by 300 ft., was seven stories high, and was of brick throughout. Along the Market and New Montgomery Street façades, there were iron columns with supporting girders for spandrel-wall construction up to the second-floor level, and there were interior steel girders supported on brick walls; but in no sense was the building a steel-frame interior, for the interior walls throughout were of common brick.

This building—the largest brick hotel in the world—was almost uninjured by the earthquake, excepting for a few minor cracks, but was, of course, completely gutted subsequently by the fire. An examination of the ruins showed that the brick walls were in excellent shape—straight as to alignment, and without other fracture than was obviously caused by the fire—and the construction was unimpaired at all the salient points of the building. The bonding, at the corners and at the points where the interior walls intersected each other or the main walls, was perfect; and this fact is mentioned especially because, in all steel-frame structures, with brick exteriors, the bonding of the front wall to the side walls was almost invariably cracked.

Many of the steel-skeleton buildings were branded by Nature's condemnation marks, in the shape of *X*-cracks, about midway

Mr. Carlin. of the height. These were due to the flexibility of the structure, thus permitting the foundations to move coincidently with the shock and before the superstructure; and, being followed by the latter, in an effort to regain equilibrium, produced great stresses about the middle third.

The Foljer Building, on Howard Street, was of brick exterior and wood interior, and was left standing after the earthquake, the fire not having reached it. As was generally the case in buildings of any description, the chimneys suffered and a small portion of the parapet wall, but otherwise the building was not damaged; and yet, at this very site, the street in front of the building had settled about 2 ft. The California Electrical Works Building, on Folsom Street, was of similar construction, and, barring the usual roof disturbance, no damage was done by the shock.

The foregoing is positively and negatively in favor of ordinary brick buildings. It seems to the writer that a most important point with reference to building construction in earthquake countries has not been emphasized sufficiently, or has failed utterly of consideration, and that is to treat the subject as a problem akin to one of buoyancy in water, and, in the design of building construction, to apply the principles of flotation. A familiar example of lack of stability in marine architecture is, of course, a small craft of little draft or beam in a heavy sea. To obtain rigidity, the displacement of a craft is increased, and, for this purpose, lead is added to her keel. Now, the undulating action of an earthquake is not unlike ordinary wave action, and should be treated in the same way. If, therefore, the foundations are designed so as to be sufficiently heavy, and are made a unit, with reference to the superstructure, the building will either safely ride the earthquake wave, or, if the superstructure is not of flexible construction, it will move with the foundations, just as a ship does, and as a unit.

The Post Office Building well illustrates the necessity for unified foundations. This building was on piles, and was located in the Mission District, which is to a great extent "made land." Although the structure was of heavy construction, it was practically supported on stilts, and the earthquake wave motion moved through this forest foundation, following and developing lines of least resistance, with consequent promiscuous racking all over the building. At the four corners the cracks were most pronounced, exactly at the angle where the heavier portions of the building joined the lighter contiguous connections. These four flanking corners were on unit pile foundations, as regards themselves, and moved independently with reference to the remainder of the building. Had this entire building been piled uniformly, and—more important still—had the foundation been capped with a solid layer

of concrete over the entire site, and of sufficient thickness and depth to make the resulting mass a unit, it is believed that the structure would have withstood the shock, and would have ridden safely the petrified waves, which now appear so clearly in the immediate vicinity of this building. Mr. Carlin.

In recent years, it was necessary to design a special method of foundations to meet the soil conditions in Chicago. This consisted of steel grillage embedded in concrete, and was called the "Chicago method," or "floating foundation." This, of course, has been superseded by caissons carried to rock.

It seems to the writer that San Francisco, in its reconstruction, will be forced to accept the "floating method" of foundation where it cannot reach bed-rock economically with a unified foundation, and that it should also use a rigid type of superstructure. Obviously, this will be obtained best by heavy masonry construction, like that of which the Palace Hotel structure was a type. These suggestions are not advanced on the score of either economy of space or of construction, but they seem to be best adapted as methods of building in regions of seismic disturbance.

CHARLES E. GOAD, M. AM. SOC. C. E. (by letter).—Conflagrations seem to be prevalent chiefly in that portion of the North American Continent inhabited by the Anglo-Saxon race, mainly owing to the "inordinate greed of immediate gain" that characterizes the construction of rapidly growing centers of population. Mr. Goad.

There are from 5 000 to 6 000 cities, towns and villages, in the United States and Canada, in continual danger of being swept by fire; thus it would seem reasonable that the question of the "Best Means for the Prevention of Conflagrations" should have serious consideration by all persons interested in the welfare of the community.

This subject is worthy of the attention of the American Society of Civil Engineers. Statistics, easily obtainable, show that the fire wastes by conflagrations (besides those caused by ordinary isolated fires) are appalling. More than 1 000 000 buildings in 10 years, says one authority; in 39 years (1866 to 1904) more than $542 000 000 of fire waste in conflagrations alone.

How often have we, who know these dangers, walked through the congested districts of our principal cities and towns, wondering why they still existed; and, on seeing "fire-traps" and "sources of danger" being hurriedly run up, do we not frequently wish for a dictator's power to stop such rash and foolish construction?

The natural foes to be met by civil engineers are taken to be water and decay. Fire, usually considered as a servant, should be regarded with more attention as a possible enemy. After a serious conflagration, there always ensues a period of "proposed amend-

Mr. Goad. ments and improvements," all too soon forgotten when the temporary panic has ceased.

The chief "physical" preventions to the spread of conflagration may be summarized as:

1.—Impervious walls and fire-resisting construction;
2.—Wide, and open spaces;
3.—Abundance of water, concentrated at the proper place, at the right moment.

The chief "moral" preventive is:

To have officials (more than one) with knowledge of how to act, and with authority sufficient to obtain immediate organized help.

Impervious Walls.—Much can be done by insisting that, in certain districts well known as liable to conflagration hazard, some walls should be constructed so as to prove a barrier to a fire that has become temporarily beyond control.

There is one good wall in the heart of London City that has already to its credit the stay of conflagrations in five instances, and the members of the fire brigade hail with delight the aid of their "old friend," when a fierce fire meets it again.

The London County Council is gradually introducing by-laws making improvements in this direction, and there is a tendency among municipal authorities to consider these matters on a more comprehensive scale.

Not only vertical, but also horizontal, fire-breaks are worthy of attention. Since the Cripplegate fire (London, November, 1897), building acts have been amended so that every alternate floor in that district must now be fire-resisting.

In the old City of Stamboul, the Turkish part of Constantinople, two stone walls, 40 ft. high, have been erected in the "Lumber District," and more are needed.

The European suburb, across the Bosphorus, Kadi-Keui, consisting mainly of buildings of wooden construction, appears to have laws which compel the construction of a brick wall, without openings, on one side of a wooden building. While it looks curious, and somewhat absurd to a chance visitor, to see a dead brick wall on one side of a wooden building, with nothing but vacant fields for 500 ft. distant, yet had such a law been enforced in communities of wooden structures on this Continent, before buildings became grouped in closely congested areas, the aid to the prevention of conflagrations would be obvious.

Wide, Open Spaces.—After every conflagration, efforts are made to increase the width of streets, and to open wide, new thoroughfares, but, as a rule, "vested interests" prove too apathetic, and nothing is accomplished.

Mr. Goad.

After the Great Fire of London (September, 1666) Sir Christopher Wren urged the construction of wide thoroughfares, and prepared a plan, the adoption of which would have been of immense benefit; but he was too far in advance of his age, and his plans were not carried out.

St. John's and Carbonear, Newfoundland, are instances where "fire-breaks" of great width, but of too limited an extent, were adopted after serious fires.

Our forefathers in New England and in the Maritime Provinces usually managed to allow sufficient space between wooden buildings in order to prevent conflagrations; but now-a-days the greed for immediate gain is such that fire-traps are allowed to be run up in close proximity to each other, and the danger is overlooked, or, if considered, is merely inadequately guarded against by fire insurance.

It would be well within the powers of hundreds of towns and villages that are springing up all over the country, to forbid the erection of wooden buildings within 50 ft. of one another, until such time as proper fire-resisting walls were built to intervene between them. The same rule should apply to suburbs of cities consisting of wooden buildings, wide thoroughfares being made wherever practicable, the widening of all principal streets being encouraged, even if at an apparently extraordinary expense.

Dynamiting Buildings.—This method is considered to be poor policy in staying a conflagration, as the shattered, disintegrated mass is the more easily ignited, and windows, hundreds of yards away, are blown in, affording ready access to burning embers.

Abundance of Water.—A lesson to be learned from the record of nearly every conflagration is, that the control of the water supply should be such that, in time of need, certain sections could be supplemented by an increased volume of water, and that it should be practicable to cut off each section in which the firemen have been driven from the hydrants without having had time to close them.

The old-fashioned system of underground water cisterns had many advantages, though, where water-works have been installed, these cisterns have generally been abandoned as unnecessary. In the case of failure of water systems through earthquake, accident, or malicious interference, however, the supply from cisterns would be an important factor in the fight against a sweeping conflagration.

Ponds, or small artificial lakes, in central squares, while beautifying the city or town, would also serve a useful purpose in times of conflagration. A small quantity of water judiciously used can be made to accomplish great results; this is known from actual experience.

Mr. Goad. *The "Moral Preventive."*—The moral preventive is to prepare for war in times of peace—to have men trained to meet a disaster—not merely one chief, who (as in many instances of late) may be incapacitated by accident, or who may be absent, but several. These men should be trained to work together, and should have a thorough knowledge of the water systems and of possible fire-breaks; they should be always in readiness to take action at the time, instead of, as is now too often the case, each one acting in such a manner as to retard the efforts of the others by want of knowledge and lack of previous instructions.

The managers of fire insurance companies frequently do good work in preventing conflagrations, though they are usually roundly abused for increasing the rates; they would deserve commendation if they not only would raise the rates, but would also make them prohibitive, or if they would decline to insure in districts which seem designed to foster the spread of conflagrations. This is now actually done by some managers, yet the majority will still accept such risks, and assist in the continuance of careless and "tinder-box" construction.

The British Fire Prevention Committee, founded in 1897, composed of architects, civil engineers, district surveyors, etc., in Great Britain, is doing valuable work in inculcating better methods of construction, with special reference to resistance to the spread of fire; its reports (now numbering 120) of fire tests, and of matters pertaining to this subject, will be of interest and service to students of fire resistance.

The National Fire Protection Association, including fifty "underwriters' organizations and engineering bodies," is doing good work on similar lines on this Continent.

The reports of the Committee of Twenty of the National Board of Fire Underwriters are valuable documents, serving to show where danger exists, and how to combat it.

While it must be acknowledged that no drastic methods of reconstruction of "congested-centers" will be tolerated by a community at the present time, yet there is hope that, by calling especial attention to impending dangers, and, by submitting remedies at every favorable opportunity, much can be done to educate the people in the science of safeguarding their homes against destruction by conflagration.

Mr. Green. R. B. GREEN, M. AM. SOC. C. E. (by letter).—The failure of the water supply for fire protection having led to a fire loss which far exceeded that from shock, it is interesting to note the respective reports of the three Committees on Water-Works, Lighting and Street Railway Transportation, and Sewers.

The first states that pipe lines and conduits, of any character whatever, are almost certain to fail when intersected by a plane of large movement; the second, that "The underground conduit system and the Edison tubing came through both earthquake and fire in very good shape;" and the third, that sewers on rocky ground were uninjured, while those on made land were wrecked, but all the wrecked sewers were old brick ones, the joints of which were of sand in place of mortar; and recommends reinforced concrete sewers. In the search for all possible emergency aids, it would seem that deep trunk sewers might furnish a supply of water for a fire-engine's suction when all regular water supplies are crippled. Every manhole would be a possible suction well. Mr. Green.

A shock that would cripple a water main under pressure might break a trunk sewer, but would probably not choke it so as to prevent ground-water from coming down and back-water from backing up from the outlet, unless there should be a tidal wave, such as would cripple even a fire-boat service. Back-water often extends in trunk sewers up through the low-level business sections of water-front cities, and could be had in new sewers by dropping a small portion of the invert below outlet level. A brick intake main extending back through the lower parts of a town would be similar, but more costly. Similarly, sewers in higher districts, known to have large inflows of ground-water, might also furnish a certain amount of aid in extreme emergencies. Bags of sand, to dam a small flow, or drop-sumps would aid in the smaller sewers.

Gates or temporary dams at the outlets of large sewers would aid in backing their flow up to the higher levels distant from the harbor front. This effect would be aided by having the fire-boats pump into the sewers after their outlets were closed.

Another aid would be obtained by having special fire protection piping in suitable piping subways. This piping, if loosely hung or laid through the subways, might pass through a shock that would cripple a pipe rigidly buried; or, if broken, would be immediately accessible in the subway and could be repaired quickly by special sleeves. This piping could be connected to the fire-boats or to special auxiliary pumping stations near the water edge.

C. E. Moore, M. Am. Soc. C. E. (by letter).—The writer has been deeply interested in earthquake strains for several years, and, in the design of structures, he has endeavored, as far as possible, to provide for such strains. Some years ago the matter was taken up and reference was made to such reports as were available, these being mainly derived from experience in Japan. In general, it has been assumed that the strains were caused by lateral motion only, and that the vertical component was too small to be taken Mr. Moore.

Mr. Moore. into account. As a rule, this is no doubt true, and the general correctness of the report of the Committee regarding the absence of vertical movement in the earthquake of April 18th is not questioned. However, from observations made by the writer, it seems to be clear that there must have been some vertical movement within a limited area in the Santa Clara Valley. As far as observed by him, the district in which this movement seems to be indicated is not more than 1 mile wide and 10 miles long. In that area there were some instances which appeared to indicate an upward impulse of a very decided nature.

Out of many cases, four have been selected as typical, and are included here:

First.—One of these is the manner of failure of the water tower at Santa Clara, which is described in the Report of the Committee on Water-Works Structures. It may be repeated here that two tanks were thrown off, and the remaining two were moved fully 20 ft. on the floor of the tower before falling. There can be no doubt of this, as it was seen by several witnesses, and it is indicated by the subsequent position of the tanks and columns. It is hardly conceivable, to one who is familiar with the situation, that this movement could have been caused entirely by sliding.

Second.—At Mountain View, at the other extremity of the area above alluded to, a timber structure was destroyed in a manner apparently similar. At Palo Alto, a few miles farther to the northwest, a structure of exactly similar construction, and about 25 ft. higher, was uninjured.

Third.—At the Santa Clara College, a large weather vane on one of the buildings has a shaft about 4 in. square. This stood in a cast-iron socket 12 in. deep, making a fairly close fit. This shaft was thrown out of the socket, neither being broken.

Fourth.—At the Catholic Cemetery at Santa Clara, a monument about 8 ft. high, the large end being 12 in. square, rested upon a base 4 ft. high. It was secured to the base by a brass pin § in. in diameter. The monument proper was thrown off the base, turned completely over, and laid with the top end next to the base and the large end (or bottom) away from it. The brass pin stood vertically 7 in. above the base, and was not bent. At the Protestant Cemetery, half a mile farther west, no such effects were produced, although several monuments were thrown down. At the County Hospital, a mile to the south, and on somewhat higher ground, the buildings were practically destroyed, but a steel structure carrying water tanks, which was erected by the writer several years ago, was uninjured. A large quantity of water was thrown out, as the tanks were nearly full, but they did not move from the floor. In their construction there was no reason for this which did not exist also at Santa Clara and Mountain View.

Mr. Moore.

At Agnews' Insane Asylum, 2½ miles north of Santa Clara, and on lower ground, near the marshes of San Francisco Bay, there was, as stated in the Report of the Committee on Water-Works Structures, a structure similar to that at Santa Clara, which was uninjured. The tanks were empty at the time, so that probably no valuable conclusion can be drawn from this fact. However, the Committee on Fire and Earthquake Damage to Buildings reports no evidence of vertical movement there, and this was the conclusion reached independently by the writer.

The axis of the strip above referred to follows along the edge of the "Artesian belt," or "Lowlands," as it is known in the Santa Clara Valley. This line passes near the Mountain View Water-Works, the Santa Clara Water-Works, through College Park, and near the Hotel Vendôme, in the northern part of San José. By following this line and going not more than half a mile on either side, one could easily locate fifty badly wrecked frame buildings. At College Park there were many, all being near this line. West of the Alameda (about half a mile from the line indicated), there was not a single case of this kind. The writer is of the opinion that the Committee's statement, that "damage to buildings * * * was confined to those faulty either in design or in construction," should be modified, as regards this locality. These buildings could not be classed as "faulty." All through the country an occasional defective building was wrecked, but in this locality the list included many good buildings, and it is these to which reference is made.

The Hotel Vendôme (a well-built frame structure) was wrecked so badly that it has been practically rebuilt, and has only recently been opened. Many residences in that vicinity fared similarly, although, half a mile farther south, such buildings suffered comparatively little. From all this it seems that the effect varied greatly in different localities, and it is not safe to conclude that, because no evidence of vertical movement is found in certain buildings examined, such movement did not occur at other points. The variation with locality appears to have been recognized in countries where earthquakes have been frequent, as one writer says: "Certain areas in earthquake localities have been marked off as unsuitable for any buildings."

It has seemed to the writer that the formation of the ground in these localities may explain, at least partly, this variation.

It would appear that any mathematical investigation of the waves would be impracticable and useless, on account of the great diversity in the character of the material passed through.

It appears, however, that the oscillations are transmitted from the seat of the disturbance by a wave movement, which must in-

Mr. Moore.

volve vertical motion, although possibly such motion may be very small. It appears also that this primary wave has a very long wave length, which renders the vertical component imperceptible. At Mount Hamilton (Lick Observatory), 20 miles to the east of Santa Clara, and at the summit of a rocky range, there was reported "no vertical movement."

At a small observatory at College Park, instruments were dismounted and no record was made. It seems, however, that, in certain localities, secondary waves are generated, and, as to magnitude and form, these depend upon the character of the ground traversed. At the Santa Clara Water-Works, there is, under a few feet of soil, a stratum of stiff yellow clay, underlaid by a stratum composed of sand and water, approaching quicksand in character. Rock is at an unknown depth, certainly as great as 1 200 ft. To the west, on higher ground, the clay stratum becomes deeper, being covered by from 30 to 50 ft. of mixed clay and gravel. The lower stratum is quite different in character, or is entirely absent.

At Agnews', the clay stratum is either absent or is covered by many feet of adobe. It appears, then, that the area referred to in the beginning is limited to a locality where a stiff clay is underlaid by a yielding stratum; and that secondary waves pertain chiefly to the surface, and do not affect the ground to any great depth.

It seems to the writer that a movement somewhat similar to that observed here may have occurred in other localities where the damage was greatest, for instance at Santa Rosa.

At Santa Clara, the first waves were from the southwest, but, before these ceased, they were crossed by waves from the northwest, these, as stated by Dr. Jordan, of Stanford University, being due to oscillations from more distant points along the fault line. The crossing of waves caused a twisting movement, by which many buildings were turned on their foundations, so that the two ends projected in opposite directions from the foundations to the extent of 2 or 3 in. This occurred with many frame houses, and in the case of certain large buildings in San José.

Referring to permanent vertical displacement, as distinguished from the vertical wave component, Dr. Jordan states that along the fault line, at the summit of Black Mountain, there was a relative change of level amounting to 2 ft., indicating at that portion of the line either a rising of one side or a subsidence of the other. The presumption is, however, that this effect does not exist, to any appreciable extent, for any great distance from the fault line.

Mr. Himmelwright.

A. L. A. HIMMELWRIGHT, M. AM. SOC. C. E. (by letter).—The Report of the Committee on Fire and Earthquake Damage to Buildings was eagerly awaited by all who have to do with the construction of modern buildings.

Mr. Himmelwright.

The span of human life is short, and a disaster such as that which occurred in San Francisco will probably not be repeated in another century. The lessons which it taught, and especially the effect on fire-proof buildings, are of such vast importance that a critical, careful, and detailed investigation and report would have been a valuable contribution to our knowledge of building construction. It is to be regretted that all the photographs taken and collected by the Committee could not have been printed with the report, because such illustrations are frequently more satisfactory than verbal descriptions. It is also unfortunate that the Committee's report is so brief that, in many cases, it is necessary to indulge in general statements, which are entirely too broad, and to that extent are misleading. A detailed report which would have embraced twenty times the space taken by the Committee would have been welcomed gladly by the architectural and engineering professions.

In the design and construction of buildings of the better class, which are now made as nearly fire-proof as possible, architects and engineers are searching constantly for those materials and methods which will show the best efficiency and fulfill the purpose for which they are intended. Definite and exact information on this subject is what is desired particularly. The behavior of any given material under varying conditions, or of the same material used in different ways, would supply information of the kind sought. The fact that all materials are damaged more or less when exposed to intense heat does not interest the designer; but the fact that one material proved more efficient than another, under the same conditions, is valuable information. Instead of collecting information of this character, the Committee seems to have made an effort to generalize and classify results, which practically nullifies the usefulness of the report.

There are in the report a few statements of facts which are somewhat at variance with the writer's observations. On page 231 a statement is made that the Bush Street front of the Mills Building leaned toward the street from 7 to 9 in. Careful determination, made under the writer's direction, in the latter part of May, 1906, showed that this front leaned out toward the street only from 4 to 5 in.

The very severe condemnation of masonry-walled structures without steel frames is a little unwarranted. Where the walls were tied together and the masonwork was executed with good Portland cement mortar, neither the earthquake nor the fire caused any serious damage. The Palace Hotel and the Parrott Estate Building, at the northwest corner of California and Montgomery Streets, are two noteworthy examples of brick and stone masonry without

Mr. Himmelwright.

interior steel frames which stood the test with very little structural damage.

The writer takes decided issue with the Committee on the general statement, on page 238, that "all materials were destroyed when directly exposed to the fire for any length of time." While it is admitted, by those thoroughly informed on this subject, that the different building materials have varying relative values, from a fire-resisting standpoint, it is now an established fact that there are materials capable of resisting in a thoroughly satisfactory manner, and with inappreciable damage to the material itself, any exposures to flames and heat which are likely to occur in any large conflagration. This is an important fact which should have been defined clearly in the report.

In considering the subject of the material for fire-proof floors, only two kinds are mentioned, namely, terra cotta and concrete, and the statement is correctly made that, of these two materials, "terra cotta suffered the more"; also that, "in all cases the record of concrete is better than that of tile." No distinction, however, is made between different kinds of concrete and tile and different forms in which they were used. There is a vast difference in the fire-resisting qualities of concretes made from different aggregates, and in the case of the same aggregates used in different ways. There is also a difference in the fire-resisting qualities of hollow-tile blocks.

The writer has made an extended study of the fire-resisting qualities of different materials used for fire-proofing purposes, not only in San Francisco and Baltimore, but also by full-sized, practical fire and water tests which have been made at different times under his direction. It is well known that stone concrete, whether the aggregates be of granite, sandstone, limestone, or trap rock, suffers from dehydration when exposed to heat, and for that reason is less desirable as a fire-proofing material than concrete made from aggregates which do not contain "water of crystallization." It has been demonstrated that stone concrete, when exposed to a temperature of from 1 800 to 2 000° fahr., for a period of 4 hours, will be affected by dehydration to a depth of from 3 to 4 in., and the strength of the concrete will be seriously impaired to that depth. When the concrete has been damaged by dehydration, the aggregates break and disintegrate, and the bond between the aggregates and the cementing material is lost, so that an ordinary fire-stream, under 60 lb. pressure, will easily abrade the under surface, and wash it away. If the concrete is allowed to cool after being heated, and without the application of water, the dehydrated surfaces can be picked apart by the fingers, or easily removed with a small tack hammer, to the depth to which dehydration has taken place. All natural rock con-

tains moisture, and for that reason concrete made from any variety of stone aggregates will be dehydrated when subjected to heat, and consequently is unsuited and poorly adapted for use as a fire-proofing material in any position where it is likely to be subjected to high temperatures. Mr. Himmelwright.

The fundamental principle in a concrete for fire-proofing purposes is the use of aggregates which do not contain moisture, but which, when made into concrete, possess sufficient strength to fulfill the requirements. Steam-boiler cinders, crushed furnace slag, crushed brick, and "tetzlonti" (a light, porous, lava rock occurring in the vicinity of the City of Mexico) are the materials best suited for fire-resisting concretes which have been found to be commercially available. Concrete made from these materials, with very little ramming, to secure lightness and porosity, has been shown to possess excellent and satisfactory fire-resisting qualities.

Another important feature of a concrete which is to resist heat is the presence of voids. A "full concrete" in which the voids are filled solidly with cementing material is desirable from the standpoint of strength, but this feature detracts very largely from its fire-resisting qualities. The ordinary requirements for strength in building construction, where these materials are used for fire-proofing purposes, are amply fulfilled by a concrete which possesses a large percentage of voids, so that their presence is a decided advantage in ordinary fire-proof floor construction.

The different forms and the principles involved in the construction of concrete floors have also an important bearing on the efficiency and fire-resisting qualities of the different methods. In all recent large conflagrations, the segmental arch, in which all the material is used in compression, and in which the strength is independent of light metal elements, proved to be in a class by itself, and developed fire-resisting qualities and strength vastly superior to any of the flat-slab methods in which light steel reinforcing elements were used in tension.

In all the flat-slab methods in which steel tensile elements are used the economical position of the reinforcing metal is invariably near the under surface of the slab, and as far away as possible from the neutral axis. When located in this position, the nearer it is to the under surface of the slab, the more it is exposed to the effect of the heat when attacked by fire; and, when thus exposed, the reinforcing metal has very little value as a load-sustaining portion of the construction.

When the reinforcing metal is covered with 1 in. of stone concrete, which is the maximum thickness of covering usually provided, experiments show that the metal is heated to a temperature of about 800° fahr. in about 1 hour, and to 1 300° in from 3 to

Mr. Himmelwright.

4 hours, when subjected to a uniform temperature of 1 800° fahr. At 800° fahr. reinforcing steel loses approximately 26% of its strength, and at 1 300° its strength is practically nil.

The flat-slab methods in which steel is used in tension are, for the above reasons, vastly inferior, for horizontal-load sustaining construction, to the segmental arch methods. In wall and partition work the use of metal reinforcement near the center of the section is much less objectionable, as the metal elements act largely as stiffening and bracing members, and the slab, being in a vertical plane, sustains its own weight and is therefore less likely to be deflected and damaged when subjected to heat.

The most elaborate and instructive fire and water tests ever made were conducted by the New York Building Department in 1896 and 1897, under the direction of Mr. Gus. C. Henning. These tests were made scientifically, and are on record in the Department. A brief *résumé* of three of these tests will be interesting in connection with this discussion. On December 23d, 1896, a stone-concrete, flat-slab, floor section, 11 by 15 ft., was tested by fire and water. The concrete floor was supported by two steel beams at about 4-ft. centers. The reinforcing metal consisted of bars at 16-in. centers, the under side of the bar being about $\frac{5}{8}$ in. from the under surface of the concrete floor slab; the section of the bar was $\frac{1}{4}$ in. by 2 in., the 2-in. dimension being vertical. The concrete was mixed in the proportion of 1 bbl. of Dyckerhoff Portland cement, $2\frac{1}{2}$ bbl. of sand and 5 bbl. of small broken blue stone passing a sieve of 1-in. mesh, with 12.8% of water in bulk. Rock wall plaster, mixed in the proportions prescribed by the manufacturers, was then applied to the under side of the flooring to a depth of about $\frac{3}{8}$ in. A second coat of this plaster was applied later, making the total thickness of the plaster from $\frac{1}{2}$ to $\frac{5}{8}$ in. A white finish was then applied in the usual manner, representing the usual conditions in finished buildings. This floor formed the roof of the test structure built for the purpose, and was located about 10 ft. above a grate, of the same area as the floor, on which a hardwood fire was maintained to produce the temperatures. The fire was started at 10.30 A. M., and the following temperatures (as recorded by a Uhling and Steinbart pyrometer) were produced at the times set opposite:

Time.	Temperature, in degrees, Fahrenheit.	Time.	Temperature, in degrees, Fahrenheit.
11.13 A. M.	1 775	2.00 P. M.	1 950
11.30 "	1 850	2.40 "	2 100
12.00 M.	2 050	3.00 "	2 200
12.30 P. M.	2 000	3.30 "	2 100
1.12 P. M.	2 100		

Mr. Himmelwright.

At 3.30 P. M., while the ceiling was observed to be in a red-hot condition, water, at a pressure of 60 lb., was applied to it with a fire-hose having a regular 1½-in. nozzle. Some of the plaster had fallen away previously, and the remainder was washed away by the water. The fire-stream also abraded the entire under surface of the stone concrete flooring to a depth of 1¾ in., wherever the water struck it, exposing all the reinforcing bars to an average of about one-half their depth.

The abrasion of the concrete to the depth stated was due, no doubt, primarily, to dehydration of the stone, and secondly to rupture and disintegration caused by the sudden cooling while highly heated, on the application of the fire-stream.

A similar fire and water test was made on a flat-slab, cinder-concrete floor on April 23d, 1907. The concrete consisted of 1 bbl. of Atlas Portland cement, 2 bbl. of clean, sharp sand, and 4 bbl. of steam-boiler cinders. The reinforcement consisted of 2½ by 6-in. mesh expanded metal of No. 10 gauge. This was laid directly on the centering, and the concrete was deposited over it to a depth of 4 in. The plaster was King's Windsor cement, applied in accordance with the manufacturers' specifications, in two coats, to a total thickness of about ½ in., approximating to the same condition as in the test previously mentioned. The fire test was started at 10.05 A. M., and at the following times the corresponding temperatures were obtained:

Time.	Temperature, in degrees, Fahrenheit.	Time.	Temperature, in degrees, Fahrenheit.
10.52 A. M.	1 800	1.12 P. M.	2 000
11.30 "	2 200	1.40 "	2 100
11.58 "	1 900	2.30 "	2 050
12.06 P. M.	2 100		

At 3.05 P. M., water, at a pressure of 60 lb., was applied to the ceiling with a fire-hose having a regular 1⅛-in. nozzle, while the under side of the flooring was observed to be in a red-hot condition. Wherever the water struck the ceiling the plaster was washed off, but only a very small quantity of the cinder concrete was abraded, barely enough to loosen in spots the expanded metal fabric, the under side of which was flush with the bottom of the slab.

A similar test was made of a segmental, cinder-concrete arch on October 28th, 1906. The concrete was mixed in the proportions of 1 part Aalborg (Danish) Portland cement, 2 parts clean, sharp sand, 5 parts steam-boiler cinders, without screening or washing, and as taken from the chutes of the New York Steam Company, and 1.35 parts water. This concrete was laid on a permanent wire centering consisting of No. 19, four-warp, two filling wire-cloth, stiffened by

Mr. Himmelwright.

$\frac{7}{16}$-in. round steel rods woven in at intervals of 9 in., segmental in form, providing for a thickness of about 3½ in. at the crown of the arch, the concrete being placed in position without ramming and by simply patting and smoothing the top surface with shovels. The plastering consisted of ordinary lime mortar furnished by the United States Mortar Supply Company, of New York City, and the two coats aggregated $\frac{5}{8}$ in. in thickness. One entire bay, or one-third of the under side of the floor area, was left unplastered so as to expose the concrete to the direct action of the flames. The fire test was started at 10.06 A. M., and the following temperatures were obtained at the times set opposite:

Time.	Temperature, in degrees, Fahrenheit.	Time.	Temperature, in degrees, Fahrenheit.
11.14 A. M.	1 700	1.06 P. M.	2 100
11.35 "	1 900	1.45 "	2 050
11.55 "	2 050	1.57 "	2 125
12.16 P. M.	2 000	2.18 "	2 130
12.30 "	2 150	2.55 "	1 950
12.43 "	2 300		

At 3.06 P. M., water, at a pressure of 60 lb., was applied to the ceiling with a fire-hose having a 1⅛-in. nozzle, while the under side of the flooring was observed to be in a red-hot condition. The following is quoted from records of the Building Department:

"Close examination of the concrete showed that the wire netting had been completely burned off in the naked arch. The concrete arch (unplastered) seemed not affected in any way by the fire or water except small spots washed away where water struck with great force."

The spots referred to were not more than ¼ in. in depth, and did not aggregate more than 3 sq. ft. of the entire surface.

Several months after this test was made, two 2-in. cubes, made from the concrete which had passed through this test, were tested for strength, and developed a crushing strength of 940 lb. per sq. in., which shows about the same strength as a similar normal concrete of the same age, and indicates therefore that, with the exception of a very thin layer at the actual surface of contact with the flames, the concrete was uninjured by the test.

Approximately 1½ hours were required in all these tests to develop a temperature of 2 000° fahr. This minimum temperature, with a maximum temperature running up to 2 500° fahr., was maintained for about 3½ hours longer before the application of water.

In the special structures used for these tests, the walls were of brick, 13 in. in thickness, with four flues at each corner. The intake openings below the grate were sufficiently large to induce the

necessary draft, and this was regulated by sheet-iron dampers. To produce and maintain the temperatures recorded, an average of about 7 cords of hardwood fuel was consumed, representing a total average depth of approximately 5½ ft. over the entire grate area. Mr. Himmelwright.

In the average office and hotel building, the combustible contents, which would be consumed in the case of a conflagration, consist of the wood finish, the furniture and the furnishings. If these were all removed and placed in a layer of uniform thickness over the floor area, there would seldom be an average depth of more than 6 or 8 in.; or, approximately, one-tenth of the quantity of fuel consumed in the New York Building Department tests. The intensity of the heat and the duration of the fire in the interior of fire-proof buildings depend upon the conditions of draft and the quantity and character of the fuel or combustible contents. In the San Francisco hotels and office buildings the average duration of the fire in any one room seldom exceeded 20 or 30 min. In special cases, as in supply and storage rooms, the duration of the fire was longer, but the conditions of draft in rooms of that kind were generally such that the fire smouldered and did not burn with as intense a heat. The average maximum temperatures attained in these buildings, as determined by the fusing of metals and other phenomena, ranged from 1 500 to 1 900° fahr. In certain spots where there happened to be more than the average quantity of fuel, and the conditions of draft were favorable, temperatures up to 2 100 and 2 200° fahr. were sometimes reached. These maximum temperatures, however, were not maintained for more than a few minutes in each case.

From the foregoing observations, and on account of the total quantity of fuel that it was possible to consume, it should be apparent, even to an inexperienced person, that the test by fire during the conflagration in the average hotel and office building in San Francisco was comparatively light, and approximately only one-eighth to one-tenth as severe as the regulation fire test made by the New York Building Department. The effect of the fire on the fire-proofing materials, as observed by the writer, was approximately in this proportion. It is patent, therefore, that nowhere near the endurance of good fire-proofing material is ever reached in buildings of this class.

There were certain structures, of course, such as the Kamm Building, the Sloan Building, and others, in which large quantities of combustible materials were stored. There were also certain rooms and limited areas in other buildings where larger quantities of combustible materials were stored, greatly increasing the duration of the exposure to fire. There were, however, no cases observed by the writer which were exposed to as severe a fire test as the standard tests of the New York Building Department in 1896 and

Mr. Himmelwright.

1897. There were places where a fire burned continuously for from 24 to 36 hours, but the conditions of draft in those cases were not favorable for the production of high temperatures over any considerable areas, and consequently the actual damage to the concrete fire-proofing was not as great as might have been expected. The greatest depth to which dehydration extended, in the most extreme cases observed, did not exceed 1½ in.; while the observations on cinder concrete did not disclose any damage whatever. The last remarks refer, of course, to concrete which was of good quality previous to the fire.

In San Francisco, stone concrete was used for floor construction and fire-proofing more generally than any other. Gravel, steam-boiler cinders and broken brick were also used as aggregates to a lesser extent. Cinders are not used more generally in San Francisco because the ferries and railroads, and some of the power plants, use oil instead of coal as a fuel, and comparatively few cinders are available. There were, however, a number of buildings in which cinder concrete had been used.

In the basement and cellar stories, the metal lath and plaster ceilings were generally omitted, thus exposing the under side of the concrete floors covered with plaster directly to the flames. Generally speaking, there were larger quantities of inflammable materials in the basement stories than in any of the others, and consequently the fire-proof floors directly above were generally exposed to a longer attack of heat than those of the upper stories. Then, again, there were instances, such as in the cases of the Aronson, Young, Scott, and Sloan Buildings, and others, in which there were no metal lath ceilings, or which had such ceilings only in one or two stories. In these buildings, and in the basement stories of others, there was ample opportunity for a careful and intelligent comparison between the fire-resisting qualities of stone and cinder concrete, as well as the relative effects of fire upon the segmental-arch methods and the flat-slab methods, in which steel reinforcing metal was used in tension. There are always sufficient phenomena, such as the fusing of metals, the effect on plaster, brick and stone work, the quantity of ashes, etc., etc., from which it is possible to glean a very good idea of the duration and intensity of the heat generated. The inexperienced investigator, however, is very likely to be led to erroneous conclusions by superficial observation. In both Baltimore and San Francisco there were buildings in which the concrete fire-proofing appeared to be damaged considerably by the fire, and many intelligent architects and engineers were misled by the appearance of these concretes after the fire. Very few took the time and trouble to seek out a spot where the material had been subjected to very little or no heat and examine the material to ascertain its actual con-

dition before the fire. Wherever concrete fire-proofing has been apparently damaged by a normal attack of fire, such as could be expected in a hotel or office building, the writer has invariably found, on close investigation, that the concrete was of poor quality, and that this fact has been thoroughly disclosed and emphasized by the fire. Mr. Himmelwright.

The conclusions arrived at by the writer as to the relative fire-resisting qualities of the different materials used for fire-proof floors are set forth in considerable detail in his report on the disaster, entitled, "The San Francisco Earthquake and Fire, 1906," a copy of which is in the Library of the Society. These may be briefly stated as follows:

The segmental cinder-concrete arch, in short spans (8 ft. or less), where the concrete was originally of good quality, developed the best fire-resisting qualities and strength. This material and this form of using it proved vastly superior to any other used for fire-proofing purposes. This method was used in the Hotel St. Francis, and in the ground floor of Haas' Candy Factory, at the corner of Mint Avenue and Jessop Street, and not a single square foot of the floor arching in these buildings was damaged in the least by the fire.

The next best fire-resistance was shown by the short-span (8 ft. or less), cinder-concrete, flat-slab floor construction, in which steel reinforcing metal was used in tension. This method and material was used in the Merchants' Exchange, in which the damage by fire was inappreciable.

The next in order of fire-resistance was the same short-span, flat-slab method of reinforced concrete in which stone and gravel aggregates were used. This method and material was used quite extensively, the best results having been shown in the Mutual Savings Bank Building, the Bush Street and South Offices of the Pacific States Telephone Company, and many others.

The next method in the order of fire-resistance was the reinforced stone-concrete construction proper in long spans, and where rolled-steel girders and beams were generally omitted. Where this method was used, a very slight attack of fire was generally sufficient to cause the rupture of the concrete underneath the reinforcing metal, so that it fell away, exposing the metal. There were comparatively few buildings, however, in which this method of construction was used.

The material which gave the least satisfactory results, from a fire-resisting standpoint, was the hollow tile blocks, whether used as end or side construction.

In many of the hotels and office buildings a separate, flat, metal lath and plaster ceiling was erected underneath the fire-proof floors, which is in itself a fire-resisting barrier of considerable value, and

Mr. Himmelwright.

a protection that must be taken into consideration in the economical design of fire-proof construction. A well-designed and executed metal lath and plaster ceiling remained in position intact in a New York Building Department test for a period of 4 hours from the time of starting the fire test. This represented a period of 1½ hours during which the temperature was raised from 70 to 1 700° fahr., ½ hour from 1 700 to 2 000°, and 2 hours during which the temperature ranged from 2 000 to 2 300°, averaging, approximately, 2 100° fahr. This ceiling was suspended by clamps or hangers made from steel stock weighing ½ lb. per lin. ft., hooked around both sides of the lower flanges of the beams, at 16-in. intervals; it supported approximately 7 sq. ft. of ceiling surface per hanger. The furring consisted of flat bars weighing 0.6 lb. per lin. ft., set on edge and running through slots in the hangers. The metal lath was stiffened with ¼-in. round steel rods at intervals of 7½ in., and was laced to the furring bars with No. 18 galvanized-steel lacing wire at every intersection of the stiffening rods with the furring bars. These details are important, and are given in order to describe the essential features of a metal lath and plaster ceiling which will give good efficiency. In hotels and office buildings, a ceiling of this character will afford ample protection for the soffits of the floor beams, making it unnecessary to cover these separately with an additional fire-proofing material. This fact also results in a saving in the thickness of the floor construction, and for this reason is an economical feature.

A fact that was demonstrated convincingly in the San Francisco fire was the undesirability of copper wire for fastening metal lath, the much lower fusing point of copper in numerous cases permitting the metal lath to drop away from the supports long before it would have failed had steel wire been used. This fact was particularly noticeable in the James Flood Building and in the rooms damaged by fire in the United States Post Office Building. It has been the uniform practice in the Supervising Architect's Office, during the last 10 years, to specify copper lacing wire for fastening metal lath in all the Government buildings. The ceilings in these buildings, therefore, cannot be expected to develop the best fire resistance in case of an actual test.

The report of the Committee tends to create erroneous impressions as to the province of metal lath and plaster ceilings, and states that in many cases they protected the concrete floors. While this is true to the extent of their value as a fire-resisting barrier, the concrete floors, when composed of good material, were sufficiently refractory to have made a good record without the ceilings referred to. In the case of the hollow tile blocks, however, where these were protected only by plaster, in nearly every case the soffit members of

the tile blocks were cracked away, exposing the cellular spaces. The very general damage of this character indicated that this material possesses poor fire-resisting qualities. Wherever well-constructed metal lath and plaster ceilings were erected under tile blocks, as in the James Flood Building, and in the upper stories of the Spring Valley Building, they prevented serious damage to the blocks. This was demonstrated clearly by the fact that wherever the metal lath and plaster ceilings fell away in spots, the tile blocks directly above were in all cases badly damaged. The tile blocks also showed great weakness and inability to resist the impact of falling bodies on account of brittleness, and numerous safes in the various office buildings very generally fell through the tile floors into the basement, breaking holes through the successive floors under them and sometimes inflicting serious damage to the mechanical plants. Mr. Himmelwright.

On page 243 of the report the following statement is made:

"The fire shows that a cover of lath and plaster directly upon the flange, protected again by the suspended ceiling, is the best. The layer of plaster alone on the flange will not protect."

In view of what precedes it, this statement, which is typical and characteristic of the report, illustrates the difficulty in arriving at general conclusions. A metal lath and plaster soffit protection is well known to be less efficient than 2 in. of cinder concrete, held in place by metal lath, with plaster applied over that. The efficiency of the metal lath and plaster ceilings and their field of usefulness as fire-resisting barriers, have already been discussed. An actual misstatement, however, is made in the latter part of the above quotation, in that there were several notable instances where a metal lath and plaster soffit protection gave exceedingly good and satisfactory results, namely, in the Kamm Building and the Aronson Building. In both of these buildings crimped or corrugated wire lath was wrapped around the soffits of the beams, the corrugations offsetting the wire surfaces about ¾ in. from the beam. This lath was placed in position before the concrete was laid, so that the edges were thoroughly anchored in the concrete. After the removal of the wood forms, plaster gauged with Portland cement was applied to the soffits, filling the spaces between the metal lath and the beams solidly with the material and making a total thickness of approximately 1 in. of cement plaster protection. There were no metal lath and plaster ceilings underneath the soffit protection, and in the Kamm Building this method was subjected to as severe a fire as occurred in any of the fire-proof buildings in San Francisco, yet a careful examination of the building immediately after the fire did not show any failure whatever of this soffit protection. The same method was used with good results in

Mr. Himmelwright.

the Aronson Building, where it was exposed directly to the flames without any intervening metal lath and plaster ceilings.

The report of the Committee goes somewhat into detail in recommending certain methods of fire-proofing columns and other important structural members. While the recommendations given are excellent for certain conditions, the treatment of columns, like fire-proof floor construction, is a question of economical adaptation to the requirements.

The protection to the steel frame, as well as the method used in the floor construction, must in all cases be adapted to the character of the building and the quantity of its combustible contents. A building with floors finished in cement, marble, or other incombustible material, with metal-covered woodwork and only a small quantity of combustible furniture and furnishings, will not require as heavy and efficient protection for the steel skeleton frame as a warehouse, department store, or similar building in which will be stored large quantities of combustible goods and materials. A standard or uniform method of protecting structural members, to be used in all classes of buildings, would not be economical, and would result in a hardship and waste in many cases.

An important fact, which is of interest to architects and engineers, and to which the report of the Committee does not refer, relates to architectural terra cotta. When properly designed, and set in a first-class manner, the results were generally satisfactory. In many cases, however, where the cores were large and the shells of the material were less than 2 in. in thickness, it spalled and cracked under moderate heat.

Another important lesson taught with great emphasis by the San Francisco disaster was the necessity of avoiding all forms of block partitions around stairway and elevator openings. These generally proved to be weak, both in the earthquake and in the fire, and almost invariably failed. The débris from such enclosing partitions fell upon the stairways, frequently breaking them down and causing irreparable damage. The failure of the block partitions also precipitated large quantities of blocks into elevator shafts, damaging the framework and doors, and often injuring large portions of the mechanical equipment in the lower story. A reinforced-concrete partition with vertical steel members anchored to the beams, or fire-proof construction at the top and bottom, is vastly superior and preferable for enclosing openings of this character.

The writer is greatly surprised at the statement made by the Committee in its conclusions on page 242 to the effect, "that the destruction of fire-proofing must be expected, and that it will have to be restored after a fire."

Mr. Himmelwright.

There is absolutely no question that, at the present time, methods and materials are available for fire-proofing purposes which are thoroughly efficient and will fulfill the various requirements for different classes of buildings in an entirely satisfactory manner. By this is meant that when suitable methods and materials are used and adapted intelligently to the requirements of any particular building, such fire-proofing will not be damaged seriously by the combustion of the inflammable contents of the building in the case of a fire. The efficiency of first-class methods and materials has been demonstrated in full-sized fire and water tests made at different times in the past, and numerous examples of entirely satisfactory fire-proofing were to be found in both the Baltimore and San Francisco conflagrations.

To reach correct and intelligent conclusions, and make definite progress in the development of fire-proof construction, it is necessary to exercise intelligent discrimination and adopt that which is good and which has proven efficient in actual test by fire; and discard and abandon that which has failed or proved deficient. This principle was followed by the writer in his report, already referred to. By giving complete, detailed descriptions of the methods and materials of construction in the instances of satisfactory fire-proofing, some of which have been referred to in the report, such as the concrete column protection in the St. Francis Hotel, the Shreve Building, and others, and the fire-proof floor construction in those buildings where good results were shown, definite information would have been available to assist architects, engineers, and others in designing efficient fire-proof construction for future buildings. Unfortunately, the report of the Committee is absolutely devoid of such detailed information, and an exceptional opportunity for rendering a public service has thus been lost.

There is, finally, a phase of this subject which has not yet been touched upon—the commercial side. The writer's long experience in the business of contracting for fire-proof construction has afforded exceptional opportunity to study the attitude of capitalists, owners, and architects on this subject. It will no doubt be a surprise to many to learn that in more than 95% of the fire-proof buildings erected during the last five years the mistaken economy of owners and their representatives has prevented the adoption of good fire-proof construction in that proportion of buildings. In every case the difference between a poor and mediocre method of fire-proofing and a first-class and efficient method has not been in excess of from 2 to 4% of the cost of the building. As long as a cheap method or system of fire-proofing complies with the building laws of the city in which the building is to be located, and fulfills the requirements for strength, the average owner is satisfied, and

Mr. Himmelwright.

is unwilling to appropriate any additional money whatever for superior methods or materials. It is the same old story of "just as good" substitution.

When the owner, as is generally the case, has no practical knowledge of building construction, and is incapable of judging of the merits of different methods and materials, he invariably adopts the lowest-priced method or system offered, or instructs his architects or representatives to do so. One of the incomprehensible things is the further fact that the average owner, or his business representative, thinks that he is fulfilling every moral and business obligation by offering to award the work to the concerns furnishing first-class and efficient methods at the same price that the poorest and cheapest methods are offered to them. This policy and method of placing contracts for fire-proof construction is used almost without exception, even by large railroads and wealthy corporations.

In the case of all other building materials, such as stone, brick, steel, cement, etc., quality is carefully considered, and the prices are graded accordingly; but in the consideration and selection of fire-proof construction, which is probably the most important detail of a modern building, quality and efficiency have been entirely neglected up to the present time.

As long as owners and architects are unwilling to pay the small additional amount necessary to secure first-class fire-proofing they must expect results such as were shown in Baltimore and San Francisco whenever a conflagration of any magnitude occurs. It would seem, however, that the exercise of the most ordinary intelligence would prompt the owner of a valuable building to expend from 2 to 4% of its cost, in order to secure exemption from damage to the structural parts of the building, and an additional 5% for the protection of exterior window and door openings, in order to save the contents of the building from exterior attack by fire.

It is to be hoped that the recent great disasters in Baltimore and San Francisco will serve to show the great importance of recognizing merit and quality in fire-proofing methods and materials, and demonstrate the necessity, as well as the ultimate economy, of using first-class and efficient methods of fire-proofing in future buildings.

Mr. Pearse.

LANGDON PEARSE, JUN. AM. SOC. C. E. (by letter).—The writer would like to obtain information relative to concrete exposed to the continued heat of a furnace, or to hot flue gases, as in a chimney. Such information would throw some light on the wearing qualities of concrete as a fire resister. In the design of a flue for the new Scioto River Pumping Station at Columbus, Ohio (under the direction of John H. Gregory, M. Am. Soc. C. E., Engineer-in-Charge), there came up the question of exposing a concrete face to flue gases. A search of the literature on the subject gave little information,

and finally it was decided to build the flue of a special concrete, using brickbats instead of crushed limestone for ballast, the flue to be lined inside with one thickness of fire-brick, and covered outside with pressed brick to match the wall of the boiler-room. This flue was 4 by 12 ft. inside, and cared for Babcock and Wilcox boilers having a total capacity of about 1 200 h.p. Mr. Pearse.

The writer would also like to point out the utility of reinforced concrete columns built up of structural shapes securely riveted together, the steel being calculated to carry all the dead load and the concrete all the live load. If suitable connections be devised to catch the floor and beam rods, the difficulty of filling the column with a mass of horizontal rods can be avoided. Such a detail was worked out for the new McGraw Building in New York City. Or steel columns, calculated to carry all the load, dead and live, with a concrete cover and filling for fire protection only, may be used. Generally, architects desire small columns, sometimes asking for dimensions which are impractical in reinforced concrete as commonly designed unless very high working stresses are allowed.

The use of steel column framing permits the use of smaller columns, puts the metal in far better shape to resist flexure, and removes all danger of poor columns caused by pouring a deep column packed full of round rods.

C. DERLETH, JR., ASSOC. M. AM. SOC. C. E. (by letter).—The report of the Committee on the Effect of the Earthquake on Highway Structures is meager, as Mr. Wadsworth has pointed out. The bridge near Point Reyes, in Marin County, was not the only highway structure injured. In the southern part of the earthquake belt two damaged highway bridges came to the writer's notice; one in Watsonville and the other near Salinas. The bottom of the Salinas Valley is a marshy district of river deposits, of small elevation above sea level. Although the region is at a distance of from 10 to 20 miles from the fault trace, there was much differential surface movement, distinct from elastic vibration. In many places along the river banks the loose surface materials were shaken and dislodged. For a considerable area the deposits on the south bank were shifted about 6 ft., and the ground moved northward into the river. In many places large sink holes were formed. From the Salinas highway bridge to Spreckels, a distance of about 1½ miles, the surface of the ground was scarred so much that the roadway became impassable for vehicles. Fig. 1, Plate LV, shows the south abutment of the Salinas bridge. The ground at this point moved about 6 ft. into the river, careening the pile foundation, as shown in the photograph, without seriously injuring the trusses. The bridge consists of two spans, with inclined upper chord trusses and wooden compression members, as shown by Fig. 2, Plate LV. Mr. Derleth.

Mr. Derleth. A 3-in. oil pipe line which crossed the bridge was ruptured on the south approach, one length of pipe being bent in the form of the letter **S**. The northern approach to the bridge was not affected. The northern bank of the Salinas River exhibited little disturbance near the bridge.

The bridge at Watsonville was distorted in a similar manner, due to the shifting of the bank deposits. Neither structure was seriously damaged. Apparently, it takes a large surface motion and severe vibration to collapse structures of this class. Neither bridge could be considered strong; in fact, they are both somewhat decayed. Heavy earthquake vibration without surface faulting will not destroy strong, rigid structures designed to satisfy the properties of elasticity and yielding. A first-class steel-framed building is an example. Similarly, an earthquake will not cause the collapse of a weak frame composed of timber, considerably loose, yet connected at the joints, provided the structure is free from concentrated loads. Such "rattle-traps" are lacking in stiffness and elasticity; but they can yield without breaking to pieces.

A number of discussors have criticized the conclusions of the Committee on Fire and Earthquake Damage to Buildings, because the Committee gives a very low rating to the earthquake efficiency of brick and blockwork structures. The conclusions of the Committee, the writer believes, will be sustained if one differentiates carefully between the terms "yielding" and "elasticity." The remarks on highway bridges (in the preceding paragraph) attempt to emphasize the difference in meaning of the two terms. In the writer's judgment, the amount of damage done to buildings throughout the earthquake belt is essentially a question of design, and not so much a matter of poor construction. Near the fault line, upon soft deposits and artificially made land where the ground was moved unequally, human structures of necessity collapsed or were seriously injured unless they happened to be very favorably situated. It is futile to use examples from these localities in discussing the earthquake effect on buildings and other structures for the purpose of concluding what are advantageous principles of design. Examination must be restricted to structures resting upon ground which did not move differentially. That is, one must consider buildings which were subjected to severe vibration, but in which the foundation did not move unequally.

Structures were thus most severely shaken upon the bottom lands of the valleys containing Santa Rosa in the north and San José in the south. In both these regions brick structures were injured to a surprising degree. In Santa Rosa, practically every brick building in the business district collapsed. In San José, along the main street, whole fronts of business blocks were thrown down.

FIG. 1.—SOUTH ABUTMENT, SALINAS HIGHWAY BRIDGE.

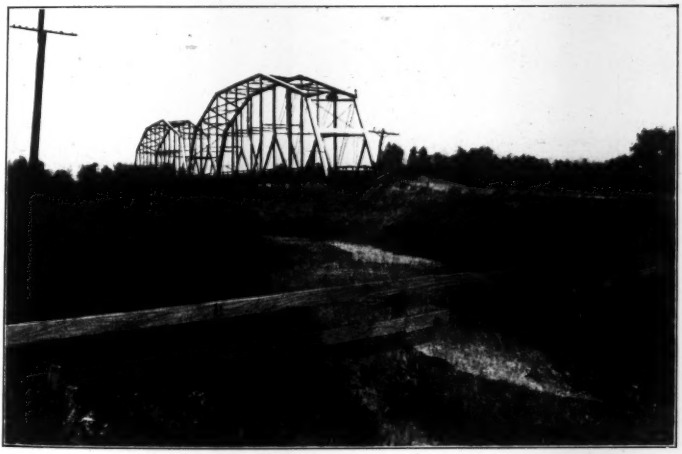


FIG. 2.—VIEW OF SALINAS HIGHWAY BRIDGE FROM THE NORTH, TAKEN AFTER THE EARTHQUAKE, SHOWING NO DAMAGE TO THE MAIN TRUSSES.

Mr. Derleth.

The city hall of that city is a mass of cracked brickwork. The brick buildings of Agnews' Insane Asylum were ruined, without exception, and more than one hundred people were killed.

Some who have taken exception to the severe criticism of brick construction have cited many cases of brick buildings which did not collapse. These are the exceptions which prove the rule. Some structures were fortunately situated or were specially and exceptionally designed. Anyone who has studied earthquake damage will at once appreciate the tremblor's fickle and peculiar variations in destructive tendency. It is not right to argue that brick buildings are adequate for earthquake countries because a hundred buildings stood in certain places. Conclusions should be drawn from the general destruction over wide areas, where geological formation allowed severe vibration without the cracking or sliding of the surface ground.

Buildings of blockwork, in general, are inadequate; the heavy stone-faced structures at Stanford University should be considered in the same class as brickwork, though perhaps not to the same degree. Earthquake destruction is measured by the type of design and the particular way in which the materials are put together. It is not necessary that all the materials in a structure shall be highly elastic, as long as they are assembled to give coherence and unity of frame, and have the ability to yield easily and not resist distortion. What is wanted in earthquake countries is design emphasis, not on the elastic components, but on the yielding frame or unit. A structure may be built of materials which in themselves are not high in elasticity, and yet these materials may be put together in such a way that the structures will have a high yielding capacity.

Ordinary Brick Buildings.—In the earthquake belt, brick walls were usually thin, the bond of brick was careless and insecure, and the walls were built with lime mortar of little strength. Very often the bricks were laid without sufficient wetting before applying the mortar. In many instances the crumbled piles of brick showed clean surfaces, the mortar not having adhered to the brick to any degree. Ordinary brick buildings usually had a timber framing within; and the floors and roofs were improperly and insufficiently anchored to the outer walls. The earthquake shook out whole sides of brick buildings because there was no provision for tying or anchoring the walls from within. The so-called fire-walls above the main cornices were thrown down in almost every case. Where buildings of this class had trussed roofs of timber, the frame was generally imperfect. The roof commonly rested on inclined rafters with no bottom cross-ties to keep the rafters from spreading. In the earthquake the vibrating peak of the roof tended

Mr. Derleth.

to drop and the ends of the rafters attempted to spread, kicking against the brick walls, and thus forcing them outward and throwing them down. Roof timbers too generally acted like battering-rams, acting upon the tops of brick walls incapable of withstanding the thrust. Many brick buildings were built like weak empty boxes, with no adequate provision for transverse stiffness. In the vicinity of San Francisco brick structures should never be built with lean lime mortar; 1 part of cement to 4 or 5 parts of lime mortar should always be prescribed; at least every other joist of the floor system should be anchored to the outer brick walls; the roof, similarly, should be bolted to the walls, and the main roof trusses should have sensibly designed lower chord tie rods; for their resistance to spreading, the trusses should not depend on the lateral stiffness of the side walls of the building.

The writer is convinced that of all the types of building construction, brick and blockwork is least adapted to the insurance of a yielding capacity for the structure as a whole, and, according to his ideas, the yielding capacity and therefore the probable safety of buildings are rated in the following order, the first being the highest:

1.—Steel frame;
2.—Wooden frame;
3.—Reinforced concrete;
4.—Brick and blockwork construction.

Brick and stone buildings will continue to be built in earthquake countries. No member of the Committee on Buildings proposes to argue that they should be eliminated in California. They must and will be used. It is clear, however, that they should never be of great height. Their foundations should be secure, and they should be built with intelligent design and construction.

Earthquake Stresses.—Many engineers with whom the writer has talked appear to have the idea that earthquake stresses in framed structures can be calculated, so that rational designs to resist earthquake destruction can be made, just as one may allow for dead and live loads, or wind and impact stresses. An attempt to calculate earthquake stresses is futile. Such calculations could lead to no practical conclusions of value. Earthquake movements may be divided into three classes:

1.—Movements at or in the immediate vicinity of a fault line or crustal rift;
2.—Secondary movements in soft deposits, such as river formations, swamps, artificially made or filled ground;
3.—Vibration in hard, solid and coherent deposits.

Mr. Derleth.

A clear distinction must be made between two types of earth motion:

I.—Motions which result in differential movements of the ground (Classes 1 and 2 above mentioned);

II.—Vibration with no appreciable unequal movement of the ground (of which Class 3 is an example).

Where the ground was moved unequally—for example near the fault line or near the water front in San Francisco—a structure unfortunately situated, no matter how well built, had to be more or less ruptured. Where a building was only vibrated, it remained unharmed in proportion to its yielding abilities.

Earthquake forces and their induced stresses may be said to stand in the relation of what are termed active and passive forces or stresses.

"If the forces which balance each other in or upon a given body or structure be distinguished into two systems, called respectively active and passive, which stand to each other in the relation of cause and effect, then will the passive forces be the least which are capable of balancing the active forces, consistently with the physical condition of the body or structure."

This is Rankine's definition. As an illustration: take the example of a redwood tree 6 ft. in diameter, the trunk of which was split, owing to the fact that it was situated right on the fault line. The ultimate shearing stress of redwood timber is known. Knowing the size of the tree, it might be assumed that the earthquake force was a given or calculable quantity just equal to the total shearing force necessary to split the trunk of the tree in question; but if that tree had been a mile in diameter, it would have been sheared just as easily. In the earthquake, the rocks cracked for a distance of 300 miles, and they could have been sundered for 600 miles if Nature had felt so inclined.

In short, the effect of an earthquake in rupturing a structure depends upon the strength and size of the structure. The amount of earthquake stress called into play is directly proportional to the resistance offered. Other things being equal, the more yielding a frame, the less its resistance, and the smaller the destructive earthquake stresses induced. If one structure is twice as strong as another, the earthquake will have to exert twice as much force, or do twice as much work to rupture it; the rupture will be produced with equal ease if each structure offers sufficient resistance to its deformation. The amount of force exerted by the earthquake will be measured exactly by the strength and resistance which exist in the structure; if a building is half as strong as it might have been with better design, the earthquake will only have to exert

Mr. Derleth.

half that much force. An earthquake will do whatever it is allowed to do. One should aim to have the vibratory effect as much like that of a tuning fork or rod as possible; and as little like that of a vibrating chain. Loose-jointed motion, like the shaking of a chain, caused a great amount of damage to structures otherwise on firm foundations. Connections should be rigid. The writer cannot agree with Mr. Stern who argues that there should be sufficient flexibility in the joints so that, in the case of unequal settlement of foundations, a building can be repaired at very much less expense than one in which no allowance for settlement has been made.

The Committee on Buildings states:

"Sufficient evidence is at hand to warrant the statement that a building designed with a proper system of bracing to withstand wind at a pressure of 30 lb. per sq. ft. will resist safely the stresses caused by a shock of an intensity equal to that of the recent earthquake."

For bridges and towers, engineers usually provide diagonal members to resist wind stresses. They would use diagonal framing in high buildings also, except for the objection of the architect. It is the writer's judgment that diagonal framing is not desirable for high buildings in earthquake countries. Such framing consists of triangular parts. Geometry teaches us that a triangle cannot change its shape without changing the lengths of its sides. Triangular framing, therefore, is stiff and unyielding, and calls forth earthquake stress to the full capacity of the diagonal wind members. If a stiff frame of triangles is subject to earthquake vibration of severity, rupture at the weakest places is extremely likely. The effect of the earthquake on the Ferry Tower in San Francisco justifies this statement. In this tower diagonal wind bracing was used. In the fifth and sixth floors of the tower the diagonal rods were either buckled or ruptured, or broken at the eye-ends, as shown in Fig. 1, Plate XXXIX. In some cases the rivets connecting the gusset plates and angles sheared off, the stress finding in each case the weakest point. The stresses exceeded the equivalent of a 30-lb. wind pressure many times. This is proven by the permanent set or elongation of the diagonal rods. In five cases, including three 2-in. square rods, it was necessary to cut off the screw ends to take up the slack. The total stretch in some cases amounted to more than 3 in.

For earthquake conditions, triangular framing for high buildings is not as desirable as rectangular framing with stiff joints and continuous members. In steel buildings, rectangular framing is best produced by substantial continuity in the main columns and

by bracing these columns with deep horizontal spandrel girders; or by more shallow spandrel and floor girders strengthened with heavy knee braces. Mr. Derleth.

Unlike the triangle, the rectangle can change its form without changing the lengths of its sides. With spandrel girder and knee bracing, therefore, the main columns, by their elasticity and continuity, can yield and vibrate to a considerable extent without endangering the integrity of the building's frame. Therefore, the writer concludes that the best type of framing in a steel sky-scraper consists in the generous use of deep spandrel girders, preferably of the latticed truss type. The new Humboldt Savings Bank, on Market Street, is an excellent example. The old Call Building has an excellent lateral framing.

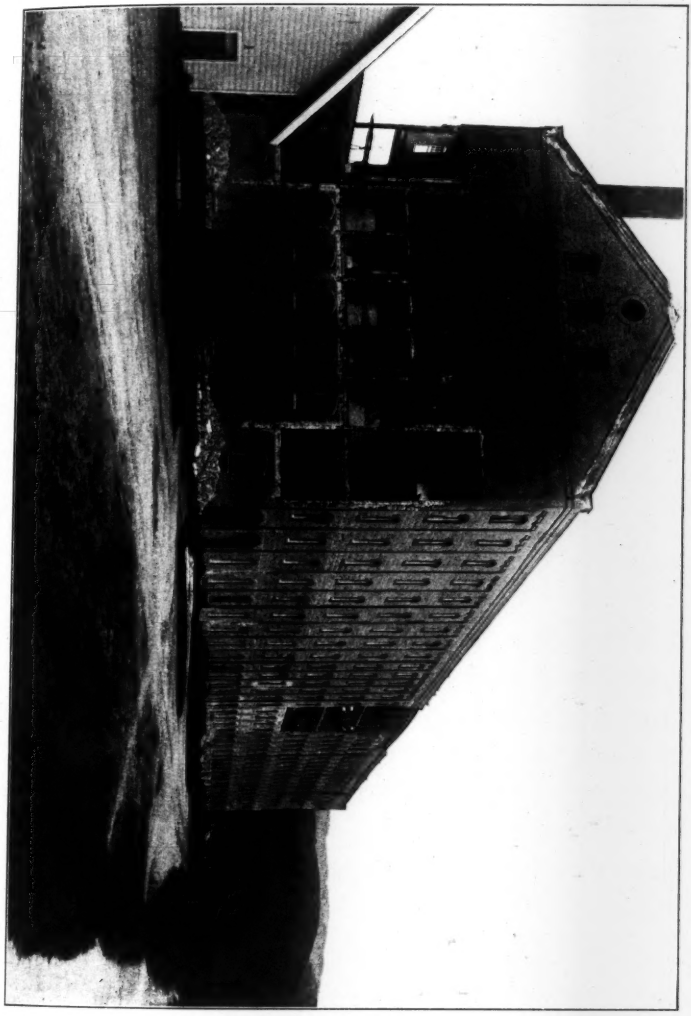
Mr. Stern proposes to limit the height of buildings of all classes to 100 ft. This is a severe and unwarranted conclusion. It is true that there was slight evidence of earthquake deformation in the wind diagonals of the Call Building, from the tenth to the twelfth stories; it is also true that in the upper portion of that building, above the tenth story, the masonry blockwork was slightly moved and loosened. In the Monadnock and Chronicle Buildings, earthquake cracks appeared in the brickwork between windows. In places the stone facing on the Flood Building and the St. Francis Hotel was spalled; but, considering all the damage done by the earthquake to San Francisco's highest Class A structures, the effect was trivial when measured in terms of fire losses. Well-built Class A structures are perfectly safe for fifteen stories. The writer sees no reason, however, for encouraging greater heights.

Mr. Stern is also ready to abandon brick and masonry walls where used as curtain walls in steel-framed buildings. There is a difference between a brick structure and a brick curtain wall. As a member of the Committee on Buildings, the writer insists that brick structures are relatively inadequate, but he unhesitatingly advocates brick and stone curtain walls, provided they are carefully anchored to the metal frame. The public is not willing to abandon brick, terra cotta, and stone for building fronts. A concrete surface is not pleasing, and the art of finishing concrete is still in its infancy. Brick and stonework in curtain walls can be reinforced just as well as concrete, and will adhere to the frame in an earthquake of even greater severity than that of April, 1906. If blockwork curtain walls crack, it is not a serious matter to replace them. In San Francisco, no blockwork which was intelligently anchored was thrown from Class A buildings.

Mr. Stern is interested to know what damage resulted to buildings of Type A which were on or near the fault line. There were no Type A structures on the fault line; but the Spreckels' Sugar

Mr. Derleth. Mill, near Salinas, was most unfavorably situated on the shifting deposits of the Salinas River. The report of the Committee on Buildings calls attention to the throwing down of 13-in. brick curtain walls on that structure. The main building, about 100 by 500 ft. in plan, is a huge box-shaped structure with a heavy steel frame; it has brick walls supported on the frame, and arched concrete floors resting on heavy steel floor framing; the building has a pile foundation. The mortar for the brickwork is excellent, and contains considerable cement. Had this building been in Santa Rosa or San José, it would have stood the earthquake, with possibly a few slight cracks and the loss of some brick here and there near the cornices and corners; but, unfortunately, it rests on soft ground, very much like the made-ground areas of San Francisco. Considerable portions of the brick walls were thrown out of the steel frame. The long walls of the main building are supported by steel columns about every 15 ft., and are braced transversely at these columns by the interior framing, except in the middle third of the building, where there is an open space from ground to roof to give clearance to the huge machinery. In this middle third, where the long walls are not braced laterally by interior transverse frames and floors, the walls were buckled badly. The east wall was buckled concavely outward, and was cracked badly, but little of it fell. The west wall, in the middle third (Plate LVI) was buckled convexly outward, and, naturally, the brickwork was thrown down in large masses. The writer believes that this building was subjected to an earth movement from south to north, which accounts in a measure for the buckling of the long walls in the middle unsupported third. There may have been some settling of the walls also. Large masses of brick in the north wall (Plate LVI) were thrown down. All the smaller buildings were damaged to some extent. The Spreckels' Sugar Mill was well designed. It was probably subjected to a more severe racking than any other well-designed structure in the State. It was quickly repaired, and speaks for the stability of Class A construction.

To Mr. Stern the Committee's conclusions as to damage to foundations are not clear. No instance of damage came to the Committee's notice. On the fault line, a foundation naturally would be crippled. In San Francisco the permanent displacements of the ground were superficial. Mr. Bernard Bienenfeld has pointed out the custom in the earthquake countries of the Far East, where important foundations go to deep strata, preferably rock, the overlying loose material being kept away from the finished building. In earthquakes, loose surface ground is much moved, but, at ordinary depths below the surface, firm material remains stable. On the made land of San Francisco, structures resting upon pile



SPRECKELS' SUGAR MILL, SHOWING NORTH AND WEST WALLS. THE LONG WALL FACES THE WEST.

foundations were little damaged although the surface materials in the streets and the vicinity were distorted into waves. On Market Street, near the Ferry House, the cable tracks resting upon piles moved very little in comparison with the collapse of the street surface on both sides of the car tracks. For deep and properly built foundations, there was little or no unequal settlement even where the surface materials were much contorted. Mr. Derleth.

Careful examination of all types of structures within the earthquake belt leads the writer to emphasize the following conclusions. In this discussion his remarks have been restricted to buildings; these calculations, therefore, refer to buildings only, and particularly to brick and stone structures; he does not expect everyone to agree with them.

1.—Cornices and top walls of buildings, especially of brick and stone structures, should not be heavy or have great projection. In the future, cornices and top walls should be anchored more securely with metal; their masonry bond should be made with care, and the cementing materials should be of high quality. A considerable number of lives were lost by falling brick and stone from such sources. These parts of buildings were very generally demolished by the earthquake.

2.—Cornices and top walls of first-class steel-cage constructed buildings, where the steel framing and anchoring has been carried into the cornice work, have resisted the earthquake.

3.—Projecting brickwork and fire-walls, of ordinary brick and stone structures, have proven themselves an abomination, especially where lime mortar was used; with cement mortar, the destruction was nearly as great, the only difference being that the material fell in large masses.

4.—Brick chimneys for ordinary brick or frame dwellings should be built of weak lime mortar above the roof line. This seems to be a paradoxical statement. In a severe earthquake the brick chimney will not move with the house. It must collapse. If built with rich cement mortar it will fall in one piece and crush through the roof. If built of lime mortar it will crumble and the individual bricks will roll off the sloping roof. Metal chimneys should be studied for the part above the roof line. If brick is insisted upon, from preference or custom, the brickwork should be reinforced with steel rods and bands.

5.—Terra cotta and similar materials should not be used as profusely and boldly in the future. They are not good earthquake materials. They have been badly cracked by earthquake vibration. The north wall of the Fairmont Hotel, where it is faced with terra cotta, was badly cracked. Pressed brickwork with good mortar and good bond is a far more satisfactory material. Terra cotta orna-

Mr. Derleth. mentation on the Mills Building and many other buildings was badly spalled by vibration. The writer would not eliminate terra cotta from use, but it should be less highly ornamented, should be manufactured with greater thickness, and the hollows should be filled with a cementing matrix.

6.—Hollow tile partitions in the General Post Office were almost wholly destroyed by the earthquake. Of course, this building rested upon very treacherous ground, but the earthquake certainly did very much more damage to tile partitions in San Francisco than is generally admitted. These partitions have little strength, and are readily collapsible. They have no elastic continuity.

7.—The writer believes that hollow tile floors were disturbed less by the earthquake than tile partitions, because there was less tendency to unequal motion in the plane of the floor; whereas, partitions were subject to a greater racking motion. Tile partitions and floors do not appeal to the writer with the same force as reinforced concrete construction, especially when the question of fire-proofing is left out of the discussion.

8.—Brick buildings without steelwork, and of light construction, should have small height, three stories at most. Their bond of brickwork should be inspected carefully, and their brick walls should be tied securely to the floor and roof frames. On soft ground their foundations should be designed even more carefully, for stiffness of framing and distributing power, than those for wooden buildings. The bond of brickwork in San Francisco was proverbially bad.

9.—Cheap lime mortar should not be allowed for buildings in the congested districts of San Francisco. Cement, with just enough lime to make it workable, say 1 part cement to 4 or 5 parts lime mortar, should be insisted upon more generally, not only in the congested parts of cities, but wherever brick buildings are used. The High School Building, at Berkeley, may be taken as an example of a lime mortar building. It was most seriously cracked, especially in the second story, and hundreds of buildings damaged in this way might be named. In the light of what has happened, it is a crime to use bad bond and lime mortar for brickwork in schools and public buildings, in fact, in any building. In too many instances mortar has been applied to dry bricks.

10.—Brick buildings of greater height than three stories should have heavy walls and a sufficient number of interior cross-walls to give lateral stiffness. The Appraiser's Building or Custom House, with 36-in. walls, was unharmed by the earthquake. The Palace Hotel stood the shock splendidly, it being an excellent type of brick structure, having brick walls reinforced with iron rods. The foundations of such buildings should be designed with the greatest care in order to prevent unequal settlement.

Mr. Derleth.

11.—All over the earthquake belt one found brick and stone structures badly demolished by the falling out of main walls due to the improper design of the roof trusses. These trusses too generally were of clumsy wood construction, improperly anchored to the supporting walls, and unscientifically framed. Too generally, they were lacking in a lower chord tension member. The earthquake caused such roofs to spread and kick out the walls, which had no ability whatever to resist a lateral thrust. The High School Building, in Berkeley, is a typical example. For large school buildings, roof trusses of steel should be used. There were many cases where brick walls stood the shock, but would have been thrown down except for the tying property of the roof trusses above them. The Majestic Theatre, in San Francisco, clearly showed how steel trusses kept high walls from collapsing.

12.—Properly constructed wooden buildings, with the exception of their brick chimneys, withstood the earthquake, no matter where located in the earthquake belt, except on the fault line and in regions of the greatest disturbance on soft ground. Some frame buildings upon favorable ground collapsed on account of improper underpinning. There should be more continuity in the frames of wooden buildings, especially at the floor levels, and the underpinning should be attended to more carefully in the future.

13.—At the time of the earthquake there were entirely too many top-heavy and improperly braced brick and stone towers and steeples in San Francisco and other cities. Where they are merely ornamental, masonry towers should be discouraged as far as possible. Their ruins were everywhere to be seen in San Francisco, Oakland, Santa Rosa and San José. The Ferry Tower might have had heavier framing, although it should hardly be mentioned in this connection. Most towers are lacking in interior cross-walls and in necessary steel frames. The top of the Memorial Arch at Stanford University was lacking in this respect. Reinforced concrete, where not too boldly used, is well adapted for these purposes.

14.—Important buildings, like the Post Office, should not be placed on filled ground or treacherous ground, but when they must be so placed, they should have heavier steel framing and lighter masonry work than the Post Office. The type of building structure to be selected in these cases should be determined by the nature of its foundation site.

15.—High buildings with deep pile foundations of the proper design withstood the earthquake shock well on soft ground. Considering the nature of the material on which the Ferry Building rests, it stood the shock splendidly, because of its excellent pile foundations. The Call Building foundation represents the slab type, apparently equally well fitted for service.

Mr. Derleth.

16.—Reinforced concrete cage construction should be more respected in the future by the building laws and trades unions of San Francisco. There is no reason why buildings of this type, designed by competent engineers, should not be at least six or eight stories high.

17.—Low buildings of intelligent reinforced concrete construction are far more able to resist earthquake shock than brick and stone buildings.

18.—On soft ground, the footings of ordinary buildings too light to require pile foundations might have cellar slabs of reinforced concrete to act as units with the wall footings to give distributing power and prevent unequal settlement.

19.—High brick chimneys for factories should be built with cement mortar; and even then they should be reinforced. In many cases a lower chimney with a forced draft should be considered. A study of reinforced chimneys should be made; they would withstand earthquake shocks much better than brick ones.

20.—In the business section of the city, Class A buildings and first-class reinforced concrete structures should be encouraged at the expense of Class B structures, and the skimping of steel frames should meet with entire disapproval. Diagonal framing should be introduced wherever windows and interior passageways or openings do not prevent; better still, heavy knee-braced framing and spandrel-girder framing should receive more attention.

21.—The design of high buildings with self-supporting walls should be discouraged.

22.—Steel columns should run through more than one floor, and their splice joints should be strongly designed.

23.—Cast-iron columns should not be used.

24.—Heavy stone ornamentation should not be hung to the steel frames of high buildings, and heavy centralized supports on the first floor, as in the Flood Building, should be avoided when possible. No expense should be spared for foundations of buildings of great height.

25.—Heavy stone corner piers at the sidewalk level of steel-framed buildings should rest upon the frame, and not upon masonry walls under the sidewalk. Where such piers rested upon concrete under the sidewalk, the stonework was badly cracked; where they were supported by the steel frame, no cracks were found.

26.—Face brick should be carefully bonded to the back brick in Class A buildings. Large areas of face brick fell from the west wall of the Merchants' Exchange Building because of improper bond (clipped course bond).

27.—Reinforced concrete buildings of careful, honest, and intelligent design should be allowed to enter competition in San

Francisco. It is unfortunate that reinforced buildings have been placed in the Class B list. To the layman this implies inferiority to Class A structures. Of their types, Class A buildings are not to be considered better than buildings of first-class reinforced concrete, but the writer would not use the latter type for structures of the greatest height. Mr. Derleth.

28.—Neglecting the problem of surface appearance, Class A buildings might be designed with curtain walls of reinforced concrete, instead of using brick and terra cotta.

29.—High buildings, like the Mills Building, in which the floors are supported on a steel frame, but in which the walls are self-supporting, should not be imitated in design in the future. When such buildings are badly shaken, or when their outer walls are badly damaged, repair is difficult. In Class A buildings, where the steel frame carries the load of each floor independently, such difficulty vanishes. The Mills Building had the additional miserable feature of columns breaking joints at every floor.

J. D. Galloway, M. Am. Soc. C. E. (by letter).—In reference to the effect of the earthquake on highway bridges, the photographs on Plates LVII and LVIII are submitted. Two views of the Alder Creek Bridge, north of Point Arena, in Mendocino County, Cal., are shown on Plate LVII. This was a 125-ft. combination span, built after the writer's designs many years ago. The fault line passed directly under the bridge, and is faintly indicated on one of the photographs. The wreck gives the impression that the bridge was lifted and dropped. Mr. Galloway.

On Plate LVIII are two views of the Gualala Bridge. This bridge was built by the late Randall Hunt, M. Am. Soc. C. E., about ten years ago. Its remarkable strength is indicated by the photographs. It is stated that one end of the bridge fell 28 ft.; but it has been raised and the bridge is now in use.

The following comment is offered regarding the criticisms of the report of the Committee on Fire and Earthquake Damage to Buildings in California, of which the writer was chairman. This comment is made individually, as it is inexpedient to call the Committee together again; the chairman, however, has asked the other members to make similar individual comment, should they so desire.

The title of the reports as printed is misleading. The earthquake affected the whole State, its violent shaking being felt throughout at least 30 000 sq. miles. The fire was a San Francisco event, although Santa Rosa was also burned.

This comment will deal first with criticisms of earthquake effects and then with criticisms of fire effects, such being the subdivision of the report. Replies will be made to those points which appear to be of sufficient importance.

Mr. Galloway.

A point of some importance which will arise in connection with such a controversy as this is that of the experience of those engaged. A reasonable degree of familiarity with the subject must be demanded of those who speak with authority. The writer was engaged for several years previous to the earthquake as a Consulting Civil Engineer, and, as such, designed the steel frame and foundations of several buildings, among which may be named the Mutual Savings Bank, the St. Francis Hotel, the Shreve Building, the Kamm Building, and several smaller ones. After the earthquake he examined and reported on the Mills Building for the owner, the Kamm Building for the lessee, the James Flood Building for the Southern Pacific Railroad, and also formed one of a committee of three (not always the same men) who reported upon the damage to and repair of the buildings at Stanford University, the Catholic Academy at Menlo Park, the County Buildings at San José, the State Hospital for the Insane at Agnews, and the State Normal School at San José. Also, as a resident of San Francisco, he has been an observer, not only of the ruins directly after the fire, but of the work of tearing down and rebuilding, long after visitors from the East had gone home.

Mr. Stern notes the diversity of opinion between the Committee on Geology and that on Buildings. No attempt was made to get the committees together, and all the writer can say is that he does not agree with the Committee on Geology in respect to its conclusions on buildings.

As to the limit of height of buildings, all that can be said is that structures from 200 to 300 ft. high were but slightly damaged. The stone of the Call Building would have moved very little if there had been any tying to the steel frame other than merely resting on the spandrel girders. The terra cotta dome was of a lighter material, but each piece was anchored to the steel frame, and was uninjured.

The only brick spandrel which fell out was that at the Spreckels Sugar Refinery, at Watsonville, but in San Francisco some others worked on the girders. Most of the brick walls remained on the girders without injury.

No unequal settlement of the foundations of any important building was noticed. The tipping of the walls of the Mills Building was not due to any settlement of the foundations, as they were found to be level. This remark applies to structures on other than "made" ground. On the areas over the old tide flats, the walls of the old 3-story and 4-story brick buildings were built on "rafts" made of layers of plank. These settled, and the space between them was arched up like the back of a turtle, sometimes as much as 6 ft. between walls 30 ft. apart.

FIG. 1.—ALDER CREEK BRIDGE. WRECKED BY THE EARTHQUAKE OF APRIL 18TH, 1906.

FIG. 2.—ANOTHER VIEW OF ALDER CREEK BRIDGE.

Mr. Galloway.

The writer agrees with all Mr. Stern's conclusions except his first, which limits heights to 100 ft., and his fifth, that walls should be of concrete. Concrete walls are much better than brick to resist earthquakes, but are not as good a fire resistant.

Mr. Carlin would seem to have it that the Committee based its earthquake conclusions on what it saw in the burned district in San Francisco. As a matter of fact, little was learned in that quarter. All the evidence then on first-class steel-frame structures was negative. There was relatively no damage at all—some cracked walls and some stone displaced. As Mr. Carlin states, all other earthquake evidence in this place was destroyed by fire or by wreckers. It was by a careful study of buildings, mostly outside of San Francisco, that the conclusions regarding earthquake effects were arrived at.

This brings up that statement which forms the major objection of other critics of the earthquake report:

"It may be stated, as one of the most obvious lessons of the earthquake, that brick walls, or walls of brick faced with stone, when without an interior frame of steel, are hopelessly inadequate."

The writer is more certain to-day than when this sentence was penned, that it is the exact truth. It is most unfortunate that, in spite of the evidence, there are engineers who will yet champion the use of brick walls in an earthquake country.

Some discussion seems to be necessary to make plain the meaning of the report. Obviously, earthquakes may and do occur, which will wreck any structure known to Man. The amount of destruction outside of the limited area, where there is an actual rupture of the earth, is not directly a function of the force of the shock. The destruction is measured by the resistance which the structure offers, the time element being considered. The earth, moving in waves, is responding to a force that, as far as human efforts are concerned, is irresistible. Structures on the surface, if flexible and light, will wave with the ground, and suffer no damage. Those of an opposite character will not move without damage. A heavy mass, such as a brick building, must respond to the wave. Its mass and rigidity are the measure of its immediate resistance, and also the measure of the damage done. The greater the resistance, the greater the damage, as the force is without limit.

In a building of a certain mass, a given amount of work must be done to set it in motion, or to bring it to rest if in motion. If the building has a flexible frame, the lower parts may move, gradually communicating the motion to the upper portions. Thus the earth wave sets up vibrations in the building. But if the building be non-flexible, it must all move at once or suffer rupture. The latter is the case with brick walls. The writer observed numbers of

Mr. Galloway.

cases where continuous brick walls were completely sheared 1 or 2 ft. from the ground. The foundations moved out from under the wall and the wall dropped. The mortar joint was a plane of weakness offering practically no resistance whatever. The other case of partial failure was where the wall succeeded in withstanding the shearing stress and communicated its motion which increased toward the top. Here the result was the overthrowing of gables and parapets—sometimes of whole walls. As the waves were propagated in all directions, walls at right angles to the direction of motion were cracked when not thrown down.

Brick laid in lime mortar can be deformed slightly, as mentioned by Mr. Bienenfeld. Of brick laid in cement mortar, this is much less true.

To illustrate the phrase "hopelessly inadequate," the writer would cite the wooden Howe truss bridge. Most engineers would say that, to meet the requirements of present-day railroad traffic, such bridges would be "hopelessly inadequate." They might do. It is conceivable that every steel bridge in this country be replaced by a wooden one, but it would not change the condition of affairs. Such bridges were fundamentally inadequate. Try as they would, engineers could not fasten the timbers together so that they would resist vibration.

Brick buildings may do in an earthquake country. Some of them stood up. Outside of the fire districts, most of them were injured more or less, and many were destroyed. Their inadequacy, however, is not a question of counting a majority either way. It is purely an engineering question. It is a proper consideration of the stresses involved and the resisting material. As such, the writer must insist that the non-flexible masonry, with every joint a source of weakness in the presence of a shearing stress, and with its dead, inert mass, is a "hopelessly inadequate" form of building to resist earthquakes.

In the rear of the writer's office the third-story wall of one building fell outward and upon the roof of a two-story building, carrying everything to the basement; seven men were crushed to death. Down the street, the falling walls killed a drove of cattle then on the way to the slaughter house. At Stanford University, the commission estimated that damage amounting to nearly $2 000 000 was caused in less than one minute. At Agnews, buildings that cost more than $1 000 000 were similarly destroyed, and 112 dead bodies were taken from the ruins. In Santa Rosa, more than 100 people were killed, every one by falling brick walls. No one knows the number lost in San Francisco. In the face of this, it is strange that engineers will still champion this material which, analytically and by evidence, proves itself to be "hopelessly inadequate."

FIG. 1.—GUALALA BRIDGE, INJURED BY THE EARTHQUAKE OF APRIL 18TH, 1906.

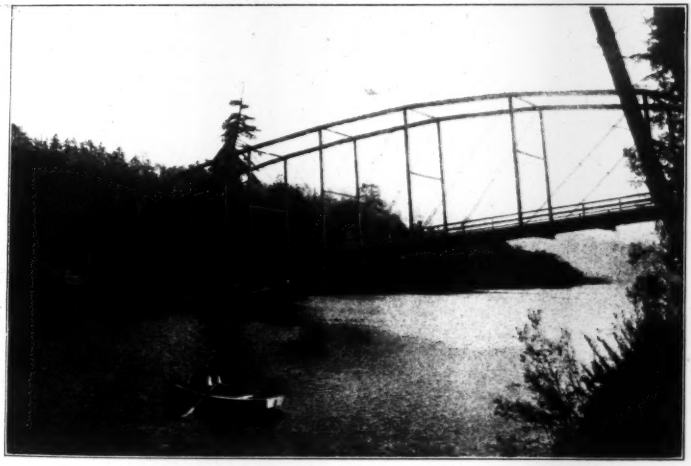


FIG. 2.—ANOTHER VIEW OF THE GUALALA BRIDGE.

Mr. Galloway.

The fire damage is next to be considered.

Mr. Freitag takes exception to the statement that the business district of San Francisco was built as other cities are. Yet he quotes a table showing that 70% of the buildings were of brick. It would be interesting to know in what way this differs from other cities. Of all the buildings in San Francisco, 2.2% were non-combustible. Mr. Freitag quotes that the Metropolitan district of New York has 2 000 such structures, out of 300 000, or a fraction of 1 per cent.

Mr. Freitag criticizes the report in that it offers a "far from satisfactory" answer to the question as to "what can and should be done to prevent conflagrations." Yet, he agrees with the recommendation—which the Committee regards as the only solution of the problem—namely:

"The only statement that can be offered is that the best insurance for buildings would be the isolation of a district containing nothing but fire-proof structures. A general conflagration would then be impossible."

After this, it is a question of the individual building being made fire-proof.

Mr. Freitag disagrees heartily with the statement that such a course would have been impossible in San Francisco. He certainly did not understand the conditions. In the altruistic days after the fire the writer was a member of the Committee engaged in re-writing the building laws. When the subject of fire-limits came up, he brought in a plan for a certain zone within the fire-limits which should contain nothing but incombustible buildings, about as outlined above. This brought out unanimous protest in the Committee, and from others, and the subject was dropped. The distant observer will ask why, with virgin ground before it, the city did not cut avenues, widen streets, and build nothing but incombustible buildings. Such comment is most superficial. The city had suffered from the greatest fire in history. Most of her industries were wiped out of existence, all business buildings were destroyed, goods burned, streets wrecked and filled with debris, sewers broken, the water supply badly crippled, and the transportation system destroyed. Comment on civic responsibility, in the face of such conditions, is mere froth. What San Francisco needs is the cheapest building possible in which business may be done, to insure the community enough to eat. The other subjects can wait.

In line with the foregoing, Mr. Goad would "forbid the erection of wooden buildings within 50 ft. of one another" in towns and villages. Let some community enforce such a rule, and it will find the people moving to some town where such a rule does not obtain.

With Mr. Himmelwright's criticism the writer wishes to take issue.

Mr. Galloway. First, Mr. Himmelwright objects to the character of the report, which gives general conclusions in place of citing a list of specific facts and allowing the reader to form his own conclusions. The Committee considered this matter, and submitted a large number of photographs, some of which were not published. The opinion was arrived at, that, in the multitude of examples, it would be worse than useless merely to cite them and allow distant readers to form their own opinions. The Committee considered that it was hopeless to describe the separate effects of the fire, and therefore decided to offer conclusions which readers could accept or reject. Personally, the writer much prefers to have, after a descriptive article, the conclusion of the writer, who, if a capable person, must necessarily have information that could not be put down on paper. By far the most valuable part of Mr. Himmelwright's report is his conclusions. The reader of the report must satisfy himself on this, and opinions will vary.

Mr. Himmelwright practically contradicts every statement made by the Committee, and states that the "effort to generalize and classify results * * * practically nullifies the usefulness of the report." Even the writer's measurement of the walls of the Mills Building, made with plumb-bobs, and verified independently by a surveyor with a transit, is disputed.

It is of little use to refute such statements, but note will be made of a few.

Mr. Himmelwright states:

"The fact that all materials are damaged more or less when exposed to intense heat does not interest the designer; but the fact that one material proved more efficient than another, under the same conditions, is valuable information."

The Committee concluded that all materials were destroyed completely. This would seem to be of interest to a designer. Their efficiencies as protecting agents were commented on, and ratings of all kinds subjected to the tests of the fire were given.

He states:

"It is now an established fact that there are materials capable of resisting in a thoroughly satisfactory manner, and with inappreciable damage to the material itself, any exposures to flames and heat which are likely to occur in any large conflagration."

The writer is positive that the San Francisco fire demonstrated conclusively that all forms of fire-proofing, as well as other materials, were and will be completely destroyed in such a fire. Attention is called to the definition of destruction given in the report. If water had been used the results would have been worse.

Cinder concrete, especially in segmental arches, is claimed to be the best, and flat slab construction is "vastly inferior." In the

St. Francis Hotel the cinder concrete of the floors was protected from injury by the hung ceilings. In the floors of the Sloan Building and Crocker Warehouse, where it was exposed to the flames, it was completely destroyed and has been removed. Mr. Galloway.

Mr. Himmelwright states that the greatest depth to which dehydration extended was 1½ in., while "observations on cinder concrete did not disclose any damage whatever." He probably bases this opinion on the columns in the lounging and dining rooms of the St. Francis Hotel, which were protected by cinder concrete. The rooms were large and with relatively little furniture; also the cinder concrete was itself protected by from 1 to 3 in. of plaster of paris in the form of scagliola. This was destroyed. In the Shreve Building the rock concrete fire-proofing of the first story was protected by scagliola in a similar way. Upstairs, the fire-proofing not protected by the scagliola was dehydrated to a depth of 4 in. and ruined. The writer has samples of rock concrete which were dehydrated to a depth of 8 in., and has the written statement of the architect that the concrete was originally first class.

Mr. Himmelwright calls particular attention to the protection afforded the steel of the Kamm Building by one cover of lath and plaster. With one or two exceptions, this building was wrecked more than any other in the city. In its reconstruction it was necessary to throw away a large part of the structural steel, which was ruined by the totally inefficient "fire-proofing."

The writer believes concrete to be the best fire-proofing material, but that it will be destroyed in a fire.

Mr. Himmelwright lays the blame for much poor work on the owner and the architect. Again the writer must disagree. That which acts most against the securing of efficient protection is the man with a "patented" system which is "superior to all others." In the lack of experience, the architect has relied upon such, and the destruction at Baltimore and San Francisco is the result. The writer has yet, with one exception, to meet the owner who, erecting a fire-proof building, does not call for the best in every respect.

As a reply to the critics, the writer wishes to call attention to the conclusions of the Committee on Buildings. They were the result of studies lasting nearly a year, during which many buildings were torn down and the debris removed. The conclusions are substantially correct, in as far as the San Francisco fire was concerned, and engineers who study them without bias may, it is hoped, derive some profit thereby.

At some time when the work of rebuilding has progressed a little farther and there is more leisure, the writer hopes to present to the Society some account of what was actually done to rehabilitate the various buildings, and also what was adopted in new structures. This will give some idea of the point of view taken by various men in the light of the lessons of the earthquake and fire.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 1057.

ELECTRIC RAILWAYS.

An Informal Discussion at the Annual Convention, July 10th, 1907.

SUBJECT FOR DISCUSSION:

"(*a*) What are the factors which determine the maximum economical grade for electric railways?

"(*b*) In establishing direct lines with heavy grades, under what conditions will it be found practicable to use electric locomotives and gas-engine generating stations, rather than traction by steam locomotives?"

BY MESSRS. GEORGE GIBBS, W. W. FOLLETT, AND J. T. NOBLE ANDERSON.

Mr. Gibbs.

GEORGE GIBBS, M. AM. SOC. C. E.—It is difficult to furnish material for discussion of a question having such scope as the first part of this topic, except in the shape of generalities, which are either obvious or of little use in the special case which the engineer may have in mind. The length of line, the character and density of traffic, the location and length of grades, all have important bearings upon the economies, constructing and operating, in any railway proposition. It is probable, therefore, that the proposer of the question had in mind a discussion of the factors peculiar to the method of traction, rather than those affecting the location of railway lines in general. These factors result from the method of application of the power to the trains, and from the interconnected character of the apparatus which makes up the motive-power system.

Mr. Gibbs.

On a steam railway the power plants are the locomotives; an electric road has its power system jointly in the locomotives (or the motor cars), in the continuous power conductor along the line, and in the central power-house. In the steam railway, therefore, the first cost of power equipment is fixed by the number of locomotives, independently of the location, or the length of line, or other outside considerations; with the electric proposition, the first cost of the power system is affected, not only by the number of locomotives or motor cars, but also by the magnitude of the other items of power equipment, and these items depend largely upon the grades, their length, and possibly their position. For instance, we can readily imagine a line with grades located in such a way that, with a given train interval, all trains may be ascending simultaneously; in such a case, the line equipment and power-houses must be designed to supply all trains taking power simultaneously, even if this maximum power is only needed for a small percentage of the total time. A location with the grades differently selected, even if the maximum gradient is not altered, might readily result in a much lower maximum demand at the power-house, and a consequent reduction in cost of the power system. Length and position, as well as rate of grades, therefore, are of much importance in an electric railway proposition.

The quantitative importance of these factors, of course, depends upon the character of the business; thus, where the traffic is very dense, requiring the operation of trains at short intervals, the relative location of grades is not as important as with infrequent train units, because, as the number of trains is increased, those ascending and descending the grades at any one time tend to balance, resulting in a relatively steady load on the power-house. The length of grade in an electric proposition has a peculiar importance because of the fact that electric motors have a rating in which the time limit comes in. Thus, for short maximum grades, the limit of motor capacity is in the commutation of the current; with long grades, the limit is in the heating of the motors, due to the cumulative effect of passing current through the motors, the resulting heat being not entirely dissipated by radiation.

As to maximum practicable gradients: this factor is generally a less limiting one in electric traction than in steam. For trains composed in whole or in part of motor cars, the question of adhesion may generally be dismissed, because the limiting grade, from the standpoint of safety, is generally less than that dictated by the adhesive limit. Even with locomotive trains, the problem is less serious in electric than in steam traction, because of the large proportion of the electric locomotive weight available for adhesion, and because of the facility with which trains may be double-headed

Mr. Gibbs. with electric locomotives; this arises from the fact that by multiple control two or more locomotives become in effect a single unit, and are not open to the objections of dual control, as in operating two or more steam locomotives on one train.

From the two factors, namely, the higher adhesion ratio and the selection of grade locations, it may result that, with an electric railway line, the economic grade may be greater than that possible with steam traction. For certain kinds of electric railways in a flat country, for instance the interurban trolley road, this facility for surmounting short but very heavy grades without reaching the adhesive limit is of great importance in separating steam and electric grades at crossings, without excessive cost, either for grading or for power plant.

The effect of gradient upon operating cost cannot well be discussed for the general case; in some cases the grades may have little or no effect, if they are short and the traffic is heavy; in other cases their length and location may have an important effect upon the quantity of fuel required and the economy of power-house operation. Of course, the cheaper the fuel, the less the importance of this factor in determining the line location.

Summing up, therefore, some important points to be considered in laying out an electric traction proposition are:

1.—The effect of density of traffic in averaging the load requirements at the power-house;

2.—The effect of location of grades, especially in infrequent service, in averaging load requirements at the power-house;

3.—The wider latitude in fixing the maximum grades because of the greater adhesion ratio;

4.—Limiting commutating effects on short grades;

5.—The motor heating effects on long grades;

6.—The less effect of grades on speeds because of the greater accelerating rate possible.

In order to use gas-engine stations successfully, for any electric traction project, it is necessary, of course, first, that gas engines and generators be obtainable of the proper characteristics for such work, and, second, that they will prove advantageous and economical. The form of the question appears to limit the subject to gas-engine prime movers for electric lines, rather than consideration of electric traction *versus* steam traction. In laying out the project in question, however, it is presumed that it would first be essential to determine whether electric traction of any kind would be advantageous, and then whether gas-engine would be better than steam or water-power prime movers.

A general discussion of electric *versus* steam traction, of course, opens up a vast subject, and to indicate even the treatment would

be beyond the limits of a brief opening discussion. Generally stated, for heavy grade work it may often be advantageous to adopt electric traction, not only because of economy of operation, but because of an increase in the capacity of the line consequent upon the use of exceptionally powerful electric locomotives which will enable the ruling loads to be taken over the grade without doubling; also because of the convenient application and control of electric locomotives, and because of the factors mentioned in connection with the discussion of the previous question. Mr. Gibbs.

Having determined that it will be economical or advantageous to use electric traction, it does not necessarily follow that gas engines would be the best means for producing power. Where water-power is available, and may be developed at moderate cost, on the line or within reasonable transmission distance of it, it will be found that such power will be more economical than either gas or steam. Where water-power is ruled out, and where the railroad is located within a short distance of coal mines, it may be that a steam-generating station would be more economical than a gas-engine station because of the lower first cost of the steam-generating plant.

In order that the gas-engine plant may be used economically, it is necessary that, as before stated, the machinery be available in units of proper size, that the details be worked out practically, and that they compare favorably with steam or water in first and operating costs. Doubtless these questions will be discussed in connection with the subject of gas engines, and, therefore, it may be stated here, that, from the point of view of economy of power, the gas-engine plant promises well, but, from the view of availability, there appear to be at the present time two important limitations: the first is in the small over-load capacity of the gas engine, which is disadvantageous for fluctuating railroad loads, and the second is in the fact that gas engines and producer plants, up to this time, have been developed in relatively small units. Of course, it is true that some large gas engines have been built, but they can hardly be called a commercial article yet for general application to railway purposes.

W. W. Follett, M. Am. Soc. C. E.—Referring to the electrification of Soldiers' Summit Hill on the Rio Grande Western Railroad, in Utah, to which reference has been made: Some twelve years ago, the speaker had a long conversation with General William J. Palmer, then President and principal owner of the Rio Grande Western, concerning the use of electrical power on this hill. After a thorough consideration of the subject, the conclusion was reached that the traffic was not heavy enough to stand the expense of electrification, that is, that the train movements were not frequent enough. General Palmer dropped the matter and never took it up again. Mr. Follett.

Mr. Follett. When trains are handled by electrical power, there must be a large volume of traffic, else the interest on the capitalization of the outfit, cost of keeping the lines charged all the time, including the loss of power by induction, and the expense of electrical repairs, will be so great as to outweigh the advantages. Of course, where train movements are frequent, there is probably no question as to the economy and advantages which follow the use of electricity.

Mr. Anderson. J. T. NOBLE ANDERSON, M. AM. SOC. C. E.—The speaker would mention an electric railway that is being constructed in New Zealand. The west coast of New Zealand is separated from the east coast by a long range of snow-clad mountains, in a manner somewhat similar, except on a smaller scale, to the way in which the Rockies separate the east and west coasts of the United States.

Some 12 or 14 years ago a British company, the Midland Railway Company, was organized and obtained large concessions of land for the construction of a railway to connect the coasts of New Zealand. After struggling with the problem for about eight years, the company finally abandoned the work, and the Government of New Zealand is now constructing the connection. The work as now designed, however, is quite different from what it was originally. The chief reason for alteration was that, owing to the fact that much steeper grades can now be used, the ruling grade being 1 in 25, a single-track tunnel about 6 miles long with heavy drainage grades has not been feared. The tunnel runs under a mountain ridge between 4 000 and 5 000 ft. above the level of the line that passes under it; and the rise in the tunnel is steeper, it is thought, than any of the Alpine tunnels between Switzerland and Italy. Of course, with electric plant, etc., there is no difficulty, and in that mountainous country, with the Bulwer River flowing past, there is no lack of water power. All the railways of New Zealand will eventually be run by electricity.

One other point: in his practice in New Zealand the speaker has found it expedient to use Diesel oil engines. These engines are used in the various cities in Great Britain and Europe for tramway traffic. All the tramways in Moscow are also run by these engines, but as yet they do not seem to be much known in the United States. They are known to the scientific men, however, and have been written about by Lord Kelvin. Mechanical engineers appreciate what the Diesel engine is, since it can be run with crude oil; it is an engine that requires only one attendant, and, as compared with an ordinary gas engine, has the advantage that it can be run at any desired speed. It can also be run at about a quarter or half its rated capacity, and practically as good economy results as when it is run under full load. For tramway work, running with very variable load is a very important factor, and an engine which can be run under one-third its capacity as economically as under a full load is of importance to everybody.

Diesel engines are used for the whole 24 hours of the day at a cost which, including interest on the plant, amounts, in England, to about £8 ($40) per h.p. per annum; many of them, if located where crude oil is cheap, give even better results. Mr. Anderson.

The speaker would like to say more about these engines, because the results obtained with them have justified the opinion he had formed from reports from such men as Lord Kelvin, and because they have exceeded all his expectations. In two particular cases where he used them, the speaker, owing to New Zealand being so far from the manufacturing point, took the precaution to have very clear specifications, making the manufacturers subject to heavy damages if the engines did not come up to the guaranty, and in both cases they are doing better than was called for by the guaranty.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 1058.

PAVEMENTS.

An Informal Discussion at the Annual Convention, July 10th, 1907.

SUBJECT FOR DISCUSSION:

"Will the paving materials of the present be used in the construction of the pavements of the future?"

BY MESSRS. GEORGE W. TILLSON, J. H. HAYLOW, CLIFFORD RICHARDSON, HORACE ANDREWS, GARDNER S. WILLIAMS, LANSING H. BEACH, E. M. T. RYDER, WILLIAM H. LAWTON, PHILIP W. HENRY, B. BIENENFELD, NELSON P. LEWIS, AND EDWARD P. NORTH.

Mr. Tillson.

GEORGE W. TILLSON, M. AM. SOC. C. E.—When one remembers that thirty years ago there was not a mile of the standard pavements of to-day in existence in America, is it strange that the thinking engineer wonders what new developments in street pavements will take place during the next thirty years? Will the principal changes be in the materials used, in the methods of using them, or both?

In order to speak with any intelligence of the future of any industry, it is always necessary to become conversant with its past history as well as the present conditions.

At the risk, therefore, of repeating some things which the speaker has said in other discussions of the pavement question, he will review briefly some of the conditions and experiences which have led up to the adoption of the pavements now in use.

Paris and London did not have street pavements until they had become cities of 200 000 and 100 000 inhabitants, respectively. This

might seem strange unless it were understood that wheeled vehicles did not come into general use until the sixteenth century, the transportation of merchandise in the interior being carried on by pack-horses and other beasts of burden. Mr. Tillson.

The first pavements were built of cobble stones and of irregularly shaped stones. These latter were gradually improved in shape until finally they developed into the large, flat stones now in quite general use in Italian cities. They did not, however, resist heavy traffic successfully. Pavements of the above character have been disclosed in the excavations recently made in the streets of Pompeii. These must have been laid nearly two thousand years ago. Their construction, with large crossing stones at the intersections, plainly demonstrates that wheeled vehicles were used very little, if at all.

Pavements, however, improved slowly, and in 1826 a London engineer, after investigation on the Continent, reported that the best pavements he saw were those with specially prepared wheel tracks, 3 ft. in width, and 4 ft. apart, the space between being paved with small stones. He said, also, that these pavements would be too expensive for London.

As cities became more wealthy, and traffic increased, better streets were demanded, and the rough block was better made, until the Belgian block of Europe and the oblong granite block of the present were evolved.

This would seem to state briefly the development of the industrial pavement.

Pavements of the above materials, however, are not satisfactory for modern cities. Pavements, now, must not only be durable but noiseless and easily cleaned, in order to be at all satisfactory. A few years ago these might have been considered æsthetic qualities, but now they are necessary. Attempts to satisfy these demands have produced the asphalt and wood block pavements.

An important quality of a pavement is that it shall not be slippery, and the lack of this quality has prevented many a material from being used successfully.

Time will not permit the discussion of the different experiments that have been made with such materials as iron, hay, glass, etc., or why they failed; suffice it to say that the standard pavements of to-day are constructed of stone, brick, bitumen, and wood.

The perfect pavement has never been constructed. The speaker has no hesitation in stating that municipal engineers in charge of pavements have failed to solve satisfactorily the problem presented to them. In most constructive work, any result can be obtained if sufficient funds are available. An architect can build any kind of a residence desired; a bridge engineer can erect a bridge that will satisfy perfectly the requirements of traffic, and be pleasing to the

Mr. Tillson. eye; but a pavement engineer cannot lay a pavement that will be entirely satisfactory; and, in this statement, durability is not considered, but only the public requirements. A pavement is laid primarily to sustain traffic, but, no matter how well it does this, it is not a complete success unless it can be cleaned easily and is neither slippery nor noisy. What pavement will satisfy the above four requirements? But, to go further. Pavements are laid for the convenience of the public, and should an engineer be fortunate enough to construct a pavement that would fill these conditions, if it were of such a character that it would wear out quickly and require constant repairs, it would be a failure. Any interruption of the traffic of a street interferes seriously with business, and often causes material financial loss; so that it can be said that the perfect pavement must not only have the properties named above, but must be so durable that its necessary repairs will not obstruct traffic seriously.

The proposition then is, can the perfect pavement be constructed or approximated with the materials now prevailing, or must new materials be used? When it is known that the great improvements in pavements that have been made in the last thirty years have been mainly in the use of many new materials, the latter might be expected; but the speaker believes that the engineer himself is at fault, that he has not made the most of his opportunities, and that he has not made sufficient study of the needs and wants of each individual street. He appreciates fully, too, that, very many times, reasons other than engineering determine the character of the particular material. But he believes that too often an asphalt pavement is considered simply as an asphalt pavement, a wood pavement as only a wood pavement, without any serious consideration as to just how it should be laid to meet the requirements of any particular location. Specifications are too general. By a little study the cost of a pavement on one street may be reduced sufficiently to permit improvements to be made to a similar pavement on another and more exacting street, and the result may be two pavements which are relatively equal.

One great defect often found is in the foundations. It is generally accepted that in any work of construction the foundation is the important part. This is so well understood that it needs no elaboration. And yet, in pavement construction, it is constantly violated.

In order to get the greatest durability from a block pavement, the traffic should be applied directly to the tops of the blocks. And, in the proportion that this practice is varied from, to just such an extent is the life of the block reduced. If this needs confirmation, examine a trench which has been roughly repaved, and note the wear on the individual blocks.

It can be laid down then, as an inviolable rule, that, no matter what the character of the pavement may be, it must always be laid on a firm foundation. Mr. Tillson.

It is now proper to discuss the different pavements in use, in order to learn, if possible, how they can be improved.

It is admitted by all that the first-class stone streets of Europe are better than streets of a similar class in America. The difference is principally in the blocks themselves. They are better dressed and more uniform in size. If more attention were given to the making of the granite blocks in America, the pavements would be less noisy and much more durable; and, as the noise is one of the greatest objections to stone pavements, anything that will reduce it is an important improvement. Granite as it is used becomes smooth and consequently slippery. This can be helped by making the blocks smaller; and, if they are well dressed, they can be set closely together, and so make it practicable to fill the joints with a bituminous paving composition alone, without gravel. This would also greatly reduce the noise. If automobile trucks come into general use, as they now bid fair to, the question of noise, now so important, will gradually grow less, and a pavement of well-dressed granite blocks, from 3 to 3½ in. thick, laid close together, with joints filled with some bituminous or similar mixture, will be very satisfactory for many streets.

One of the greatest objections to a good brick pavement is that it is noisy. It is often difficult, however, to tell by preliminary tests how satisfactory an unknown brick will be in a pavement, the product of different kilns often requiring different tests to enable the engineer to determine which is the best specimen. As brick is a material which is so available for many interior cities, and has given such excellent results, as far as durability is concerned, any practical change in methods which will reduce the noise will be of great benefit. As the speaker has had a limited experience with this material, he feels somewhat diffident about making suggestions, but thinks that, in addition to having the joints filled with a yielding substance, if the bricks themselves could be bedded in the same material, without too great an expense, the noise from the traffic would be materially deadened.

The construction of coal-tar pavements in Washington, some thirty or more years ago, and their development into the bituminous pavements of to-day, has made a greater change in street conditions than anything that has occurred since streets began to be improved. They filled several long-felt wants, as they were smooth, comparatively noiseless, and easily cleaned. Thousands of miles of such pavements have been laid since 1877, and their popularity is still increasing. The pavement mixture is artificial, and con-

Mr. Tillson. sequently is subject to considerable variation. There is no class of pavements that taxes the ingenuity of the expert as much as the bituminous ones. When these pavements were introduced they were constructed according to the specifications of the contractor, who guaranteed them for a term of years. This practice has been kept up, to a certain extent. Few city officials hold their positions long enough, or have sufficient facilities for studying such subjects thoroughly enough, to decide arbitrarily on asphalt specifications. The general public looks askance at specifications prepared by contractors, but what can be done? The only thing is for the official to have sufficient knowledge to determine whether the conditions are drawn in the interest of the contractor or the city. Few engineers of the country are qualified to design asphalt pavements for different conditions. That is one reason why such pavements have so often failed. They were not properly designed for their work. The one great defect has been in the foundation, but the wearing surface is of great importance. An asphalt pavement is like a carpet laid upon a floor. Both the carpet and the floor must be good, or the result is bad; a failure in either causes a failure in both.

The three forms of bituminous pavements now in use are sheet asphalt, asphalt block, and the bitulithic. Sheet asphalt pavements made the reputation of such pavements. The main objections to them are that they are slippery, and will not stand heavy traffic. Both these objections can be helped, if not overcome, by proper design. Clifford Richardson, Assoc. Am. Soc. C. E., in a paper recently read before the Society of Chemical Industry, advocated a pavement made up of a 6-in. concrete base upon which was to be laid 2 in. of a solid binder, covered with a 1½-in. wearing surface, this to take the place of the 1 in. of loose binder and 2 in. of wearing surface now in use. The speaker firmly believes that such a pavement would prove a success on many streets where the traffic now seems to be too heavy for asphalt. A proper selection of the materials for the wearing surface will undoubtedly reduce greatly the excessive slipperiness, although a hard, smooth surface must be slippery to a certain extent unless kept perfectly clean.

The asphalt block pavement is designed for steep grades. The aggregate of which the blocks are formed is coarser than in sheet asphalt, and this, together with the joints formed by the blocks, gives horses a better foothold. Good blocks will make a good pavement, and elaborate experiments are now being made, in order to determine a proper test for, and a proper method of testing, these blocks, so that their value can be determined before they are used.

The bitulithic pavement is expected to be smooth and not slippery. Its inventor started out to improve macadam, but finished by producing a new pavement. It has been used extensively and

satisfactorily during the past 5 years, but the speaker's personal experience with it has been so slight that he will not attempt to discuss it. Mr. Tillson.

Wood pavement has been used in 20-year cycles since the Forties, and has been steadily improved. At first, it consisted of blocks of any convenient wood laid carelessly on a sand base. The pine blocks of the Nicolson patent, laid on planks, followed in the Sixties. The blocks decayed and wore out unequally, and the pavement soon became in disrepute. Cedar and cypress blocks, made of fence posts, were used extensively in the Central West in the Eighties, and, in the present decade, has come the modern treated wooden block. This is a very different proposition from any of the others just described. They all failed from decay. The chemical treatment of the present blocks is expected to prevent this and make durability a question simply of wear. The modern wood pavement, when dry and first laid, is practically ideal. It is smooth, noiseless, and is easily cleaned. It is slippery, however, in moist or frosty weather. That is its greatest fault. If this objection can be overcome, it is bound to have a great future, especially if its preservative treatment is sure. It is expensive, however, and, as all lumber is increasing in value from year to year, it would seem to be difficult to reduce or maintain its cost as at present. This may possibly be done by obtaining different varieties of wood for use. It must be understood, however, that there is no material used in the constructive arts the value of which is so difficult to determine in advance as wood, and great care must be exercised in deciding to use a new variety. The kind which has been generally used is long-leaf, yellow pine, but this, while good, is expensive and is becoming more so from year to year. This wood comes from one section of the country, so that transportation expenses are high. The problem, then, is to discover, if possible, different kinds of wood, in different sections of the country, which can be used to advantage.

In summing up, then, the speaker would say that, in his judgment, the improvements in street pavements in the future are to be brought about by a different use of the materials now in vogue, rather than by the adoption of entirely new materials. That, by studying the needs of the different streets to be paved, by using present materials intelligently, both as to manner and combination, the municipal engineer can bring about results which, if not entirely satisfactory, will be a vast improvement over present practice.

J. H. Haylow, Assoc. M. Am. Soc. C. E. (by letter).—The materials used in the pavements of the future, speaking of a future of from 25 to 50 years, will no doubt be principally the materials of the present. While the pavements of to-day fall far short of Mr. Haylow.

Mr. Haylow.

perfection, it is due to faulty construction and abuse, rather than to the materials used.

Of the many thousands of miles of pavements in the United States, only a small percentage has been constructed in a scientific and substantial manner. The construction of a scientific pavement consists, first, in the thorough and effective drainage of the subsoil; and second, in laying a substantial foundation, and unless this is done a full life cannot be obtained from any wearing surface.

The greater part of the mileage of existing pavements is on streets occupied by street-railway tracks, and that portion next to the rails invariably fails first. This is sometimes caused by the faulty construction of the street-railway roadbed, or, by the track not being sufficiently rigid, and sometimes on account of the section of rail used. For instance, a tee-rail should never be used in a street paved with brick, granite block, or asphalt. With a tee-rail, the pavement has no protection whatever from the grind of the wheel flanges and steel tires, the groove in the rail being seldom, if ever, wide enough to be entirely free from the wider flanges and inequalities in the gauge of trucks, and the pavement being subject to the constant wear of steel tires.

Some cities have eliminated the narrow steel tire by ordinance, and, in the writer's opinion, the time is not far distant when all steel tires will become obsolete, their place being taken by rubber or pneumatic tires.

When the steel tire is eliminated, and when city engineers and street-railway officials become convinced that the girder rail—similar to the Trilby type—is the most economical and satisfactory rail for paved streets, and when the portion occupied by tracks is paved with brick, granite block or other material which can be laid on a sand cushion and the joints filled with tar instead of cement grout, then that part adjoining the rail will have the same durability as the remainder of the pavement. Asphalt should never be laid next to the rail.

A first-class street-railway roadbed should possess a certain amount of elasticity, but when the paving joints are filled with cement grout they become ruptured and are no longer impervious to water. The demand at present is not for new, but for a proper handling of the present, paving materials.

Mr. Richardson.

CLIFFORD RICHARDSON, ASSOC. AM. SOC. C. E. (by letter).—This subject is of very great importance, and Mr. Tillson has brought out some points of great interest, many of which the writer has been insisting upon for years, basing his conclusions on twenty years' experience with various forms of pavements in more than one hundred cities in the United States, in England, and on the Continent of Europe.

Mr. Richardson.

There is no question that the failure of most of our pavements is due to lack of foundation as much as to any other defect. This is particularly true in the City of New York, where, as far as the writer is aware, a single well-constructed pavement does not exist. In a few, where there is a hydraulic cement foundation, other defects, such as the use of an open binder course in asphalt pavements, result in lack of support to the wearing surface. It is very fortunate that Mr. Tillson, by assuming the office of Chief Engineer of the Bureau of Highways, in the Borough of Manhattan, will be able to inaugurate some much needed changes in the character of the pavements to be laid in the metropolis of the country, and will thus do away with a condition which is a disgrace to such a large city.

The writer does not see that there is any possibility of the immediate introduction of any new form of pavement, but that all improvements must be in the methods used in the construction of those forms now in vogue.

Our granite block pavements do not compare favorably with the best in England and on the Continent of Europe, and we could well imitate the form of construction practiced in the City of Liverpool. As Mr. Tillson says, our blocks, owing to the fact that the surfaces are not well dressed, are made much too large to bed satisfactorily. A granite block, when well dressed, should not be more than 3 in. square. The long blocks now in use make a surface which is far from acceptable.

A large proportion of our brick pavements, even when supported by proper foundations, shows serious defects, owing to the careless way in which the bricks are embedded in the sand or cushion coat between the foundation and the surface. When properly constructed, brick pavements are very suitable for streets of moderate traffic, although they do not possess the smoothness or the freedom from noise of asphalt and wood block.

As Mr. Tillson remarks, we are much in need of some satisfactory method of making accelerated tests, not only of brick, but of wood, asphalt block, and sheet-asphalt surfaces, under conditions approaching in some degree those which exist in actual service.

The writer cannot agree with Mr. Tillson that wood block pavements are to be, to any degree, the pavements of the future, except as luxuries, owing to the scarcity and increasing cost of lumber, the short life of this pavement, and the impossibility of maintaining it in a satisfactory form. It is almost impossible to replace an opening in a wood pavement so that it is not uneven under traffic and plainly visible. London and Paris have had the most extended experience with wood block pavements, and the

Mr. Richardson.

Municipal Council of Paris is so dissatisfied that it is endeavoring in every way to limit the area paved with wood. In London, streets which carry heavy travel have to be replaced within at least 7 or 8 years, even where 6-in. blocks are used.

The custom of using large quantities of impregnating material, as practiced in the United States, does not appeal to British and French engineers. The material used for this purpose does not increase the essential resistance of the wood to wear, and it is only necessary to use a sufficient quantity to prevent rotting, at least on streets of heavy traffic.

Blocks having a depth of only 3½ in.—which have been used of late in some of our largest cities—can hardly prove to be satisfactory. After a period of only 3 years, in New York City, they are showing signs of serious deterioration. The experience derived from these pavements will be valuable but expensive.

For many years asphalt blocks have proved extremely satisfactory on residence streets. In Washington, D. C., the writer recently inspected an asphalt block pavement, laid in 1883, which is now in nearly as satisfactory condition as when originally put down; but attempts to use this form of pavement on streets carrying moderately heavy traffic, as in New York City, have resulted disastrously. A good example of this can be seen on Fourth Avenue, between Thirty-second and Thirty-fourth Streets, in New York, where the blocks began to go to pieces within 18 months, and have been largely replaced within 2 years.

There is, no doubt, opportunity for improvement in the character of the blocks by using a cementing material less susceptible to temperature changes than that which has been used. In order to make it possible to store the blocks at the plant, and to transport them to the street without losing their shape, it has been necessary to use a much harder cement than was satisfactory under traffic. Some of the native bitumens and fluxes now available have made it possible to prepare a cement which is sufficiently soft at winter temperatures to make a block which will not disintegrate under traffic at that season, and at the same time be hard enough to permit of handling and storing in our hottest summer temperatures in the sun.

Within the last few years very decided improvements have been made in the form of construction of sheet-asphalt pavements, although these improvements, owing to the inertia of municipal authorities, have not been generally adopted. The best form of construction for a sheet-asphalt surface has recently been described by the writer, in a paper read before the New York Section of the Society of Chemical Industry, and details in regard to this need not be gone into here.

Mr. Richardson.

Bituminous concrete pavement is one of the forms recently developed to a considerable extent. When the cementing material is a bitumen of suitable character, and not coal-tar, such pavements serve extremely well on streets of light traffic. One constructed by the writer in Muskegon, Mich., in 1902, has been entirely successful after 5 years of constant use. Pavements of this description will, no doubt, be used to a considerable extent in the future, but they must be constructed on practical and not on theoretical lines. Already the large stone used in the original pavements of this type has been abandoned for smaller sizes, not exceeding 1 in. in diameter; and the quantity of fine material has been increased to a marked degree.

This form of pavement is not satisfactory on streets of heavy traffic, owing to the fact that the coarse stone is torn out very readily by the impact of horses' shoes, with resulting disintegration of the surface.

It seems to the writer that Mr. Tillson has not paid sufficient attention to the fact that that form of pavement is most desirable, especially in American cities where so many openings are made in the streets, which, while desirable in other respects, can be maintained most perfectly and most economically. It is certainly most unfortunate that our municipal engineers have done nothing toward determining the lasting properties of the various forms of pavements, under traffic of various concentration, and also the cost of maintenance of each type. For this purpose, very complete counts of the traffic should be kept in every city, in order to determine the cost of various pavements annually per ton of material hauled over them.

The writer is convinced that much better pavements could be laid by contractors if municipalities were prepared to pay a somewhat increased price. There is no economy in laying a pavement because it is cheap, if a stronger form of construction at a higher price would eventually result in a smaller annual cost.

Another unfortunate state of affairs in American cities is that the average citizen believes that when a street is once paved it is finished for a generation, or for all time, and little or no attention is given to its maintenance. In one city which the writer has in mind, at a meeting of the Board of Public Works, it was stated by a city official that an asphalt pavement was of no value, and that no more pavements of this kind were desired in that city because, after the expiration of a 10-year guaranty, nothing having been expended on maintenance for 5 years, the pavement went to pieces.

Finally, it may be remarked, what is needed in the United States is not new material for the construction of street surfaces,

Mr. Richardson. but a proper and improved form of construction of those which have been in use for the last 20 years, together with a study of the actual cost per annum per ton of traffic carried per linear foot of street. Unfortunately, data of this description are almost entirely wanting at present.

That there is a large field for improvement in the character of pavements is quite evident, and this improvement must lie with the municipal engineer, who will probably not make the desired move until the citizens demand that he shall do so. The best contractors are always in favor of doing the best work compatible with the price they receive for it.

Mr. Andrews. HORACE ANDREWS, M. AM. SOC. C. E.—The subject as announced seems rather vague. Judging from the opening paper and discussions, the question might have been altered so as to read: "In what manner will present paving methods be changed in the future, and what materials will be used?" As almost all possible materials have been used, up to the present time, it is pretty safe to say that the materials of the future will comprise those that have been used in the past.

Paving materials may be divided into two classes: inorganic, such as stone or brick; and organic (those containing carbon in chemical combination), such as asphalt and wood. The latter class is more unstable than the former, and is liable to depreciation and decay from causes which have no marked effect upon inorganic materials.

At present, under certain conditions, the organic pavements seem to give good satisfaction and to withstand the trying impact of the iron-clad hoofs of horses with little wear and with a minimum of noise. The observations of the speaker within the last few weeks indicate that in the cold climates of our northern cities, Buffalo, Montreal, Albany and New York City, asphalt is disintegrating from causes other than heavy traffic or the abuse of cuts for trenches. A chemical change, or some loss of volatile constituents, seems to be responsible for these depreciations, which are widespread to an alarming extent. Perhaps this breaking up of the asphalt is due to great changes of temperature; at all events, it is not noticeable in the City of Mexico, nor has it been observed by the speaker in Washington.

If chemists can ascertain the exact treatment necessary and the precise kind and quantity of asphalt needed for any given climate and combination of sand or broken stone, so that present uncertainties may be avoided, the continued use of asphalt will be better assured.

Asphalt blocks laid with the smallest dimension vertical, or as horizontal slabs, so as to simulate sheet asphalt, do not seem to be a successful innovation. A pavement so laid is going to pieces very rapidly in the City of Albany. Perhaps it is the well-known tendency of a slab to rock and move under traffic that facilitates its destruction, especially where cold causes shrinkage, opening of joints and brittleness of the asphaltic mixture.

Mr. Andrews.

The speaker has given some thought to the effect upon the condition of a pavement, which is due to covering the soil with a water-proof roof of pavement, thus lessening its normal supply of water from rains. Sometimes the sidewalks are also nearly water-proof, and then the water contained in the soil's interstices can only be drawn from below, and in lesser amount, by capillary attraction. It is reasonable to suppose that the normal proportion of water is reduced by a water-proof covering, and that a shrinkage of the subsoil beneath the pavement may then occur. The speaker has observed a settlement and cracking of impermeable pavement which replaced cobblestones, where the subsoil was a pure clay. Such settlement is difficult to explain under any other hypothesis than that mentioned.

A secondary question arises: "Will the traffic of the future be of the same nature as that of the present?" Within a few years there has been a rapid increase in the use of motor vehicles and the adoption of elastic tires. At present these do not materially affect traffic conditions, for horses are as numerous as ever, but who can say that this use of horses will not cease at some future day? In that case many alterations in paving methods may ensue. Already strips of brick pavement in macadam roads are proposed, in order to avoid the raveling of the latter by automobiles. Such a combination is now being adopted on one avenue in Albany.

The advisability of the comparison of the wear of pavements by some scientific process involving a census of traffic is obvious; but in taking such a census, the personal equation may be large. The observer must guess at the weights of the contents of many loaded wagons, and his guess will materially affect the results.

In reducing a census of traffic to tons per linear foot of width of carriage-way, the deduction of the street railway tracks from the width of the street produces a marked uncertainty in the result. In places, the existence of street railway tracks may add to the traffic on the adjoining portions of the street, but this is not invariably the case, nor is there an equality in this respect in all parts of the same street.

In studying certain traffic statistics, the speaker has been in a quandary as to the value of the ton used. A difference of more than 10% will exist, depending upon the use of 2 000 or 2 240 lb. for the ton. The weight of the ton used is not stated in some instances. If traffic statistics should be expressed in kilogrammes per meter of width, this uncertainty would be done away with, and international comparisons would be easily possible. The material of the vehicles' tires should be noted in the census. It would be a matter for congratulation if some well-recognized body should inaugurate a change in the metrical system, as above suggested, and should also formulate regulations for observers of traffic, with sufficient minuteness to render possible exact comparisons between different pavements in different cities.

Mr. Andrews. The speaker is of the opinion that, in the severe climates of northern cities, well-laid pavements of inorganic materials are more likely to be satisfactory than others, and he would not overlook the possibility of making an entire pavement, including the wearing surface, of cement concrete. A pavement of this description has given good satisfaction for some years in front of the Capitol at Albany, and seems to be well adapted to automobile traffic.

Asphalt and wood may be used where the expense will not prove a hardship to the taxpayers; they are in the nature of luxuries, like rugs and carpets on floors. Cities with mild climates, or where the lasting qualities of these perishable materials may be prolonged by other favorable conditions, may use these organic materials more freely than their less fortunate or less wealthy neighbors.

Mr. Tillson has referred to an improved asphalt pavement "made up of a 6-in. concrete base upon which is laid 2 in. of a solid binder, covered with a 1½-in. wearing surface." A pavement was constructed under the supervision of the speaker and after consultation with Colonel Greene, of the Barber Asphalt Paving Company, on Center Market, in Albany, in 1889, which was very closely in accord with the suggestion quoted. This pavement is still in good condition and has never been repaired.

The speaker is of the opinion that granite pavement may be improved by the use of stone showing some stratification, since better-shaped blocks may be obtained with such stone, and their wearing qualities are often excellent. Some method of improving the exactness of shape of stone paving blocks is certainly to be desired.

Mr. Williams. GARDNER S. WILLIAMS, M. AM. SOC. C. E.—The speaker would call attention to one phase of paving which appears to have been wholly overlooked, namely, the sanitary question. There is something in a pavement besides wearing qualities and costliness. An ideal pavement should be one which, with a good wearing and noiseless surface, does not give off dust, nor allow the germs which may accumulate upon it to be blown away until they can be removed by the proper agents. It is known, of course, that there is a vast difference in the surfaces of pavements in this respect and that, so far as they have gone, experiments have shown the very worst in this respect to be sheet asphalt, the best to be cedar block, and other pavements to be intermediate between these two. The speaker does not wish to be understood as saying that cedar block is the best pavement, or that it is likely to be the one adopted as ideal, but the germ-disseminating quality of a pavement is a question of prime importance, and it is a matter for surprise that none of those expert in the subject under discussion appear to have ever thought of it.

It may be said that experiments have been made by locating gelatin or agar plates at specific distances above different pavements,

under various conditions of cleanliness and wind, and observing the development of bacteria on the plates; observations of the presence or absence of dust have also been made on windy days, and such investigations have shown that the number of germs and amount of dust in the atmosphere were far greater in the case of sheet asphalt than any other, and far less in the case of cedar block.* Mr. Williams

LANSING H. BEACH,† M. AM. SOC. C. E.—In many respects, the question of pavements is very similar to that of clothes. What is suitable for one man and one occupation is not the best for another; what is well adapted for cold weather in a northern climate is by no means that which would be selected for summer in a southern latitude, and then, too, many men wear not what they would like, but what they can afford. Similarly there is no one pavement which possesses all advantages, is best for all climates and kinds of traffic, and adapted to the purse of the poor village as well as to that of the rich metropolis. There is no such thing as the best pavement for all towns and highways any more than there is a best kind of clothes for all men under all circumstances. Maj. Beach.

The first and principal use of a pavement is to keep pedestrians out of the mud and to make traction sufficiently easy to permit the hauling of relatively large loads. The result is that the first road covering is generally of the cheapest class, often not worthy of the name of pavement. Broken stone, cobble-stones, gravel, and shell have been used where such materials are obtainable at small cost. When these are not so easily procured, the community resorts to such makeshifts as the vicinity offers. Cost varies so greatly with the locality and the materials which that locality offers, that no general comparison of expense is possible. For instance, in some parts of Ohio where paving brick are made in large quantities, vitrified brick pavements have been laid in some cases at a price as low as 35 and 40 cents per sq. yd.; this is less than the cost of macadam in many cases. Even the cost of as simple a covering as macadam varies greatly, depending upon the cost of quarrying and crushing, and length and character of haul. Cases are known where all the conditions were most favorable, and the cost did not exceed 25 cents per sq. yd.; in others, not so fortunate, the cost of similar work has been more than $1 per sq. yd. Sheet asphalt varies greatly in cost, depending upon the transportation necessary to bring the asphalt to the site. In Washington, where it can be brought in vessels, the cost is generally less than $2 per sq. yd. upon an 8-in. concrete base. In the interior of the country, the cost is higher under similar specifications. Brick pavements vary less in price perhaps than those of any other material, owing to the numerous places where such brick is made and the correspondingly large number of centers of dis-

* See *Engineering News*, Vol. XLIV, p. 242, for a description of a series of such observations.
† Major, Corps of Engineers, U. S. A.

Maj. Beach. tribution. Several of the large European capitals, London and Paris, for instance, use a wooden block pavement laid upon a thick concrete base, that in London being ordinarily 12 in. deep. The woods used are largely from Australia and New Zealand, and the cost of the finished pavement is between $4 and $5 per sq. yd., usually nearer the latter figure. There are few municipalities which can afford such an expensive covering for their streets and avenues. For a small village, or one of our new towns, with their comparatively light traffic and limited means, to attempt to use such a pavement would be as consistent as for a bricklayer or an engineer to wear broadcloth at his work.

The question of cost of a pavement includes not only that of first construction, but also the money which must be spent for maintenance. A macadam roadway traveled by many vehicles becomes costly to keep in proper condition, if the stone is at all soft, and is hardly ever an economical road covering from the maintenance point of view. Some asphalt block pavements upon residence streets in Washington have not required an expenditure of a dollar for maintenance in fifteen years.

The matter of repairs to a pavement is often, if not generally, discussed as if it were one of maintenance solely, but the fact remains that the closing of plumbers' cuts, or water-pipe or sewer trenches, or other cuts, so freely made even in new pavements, is different and requires different treatment. It is one thing to renovate the whole surface, it is another to put a patch in a good pavement so that it will not show that it is a patch and be a blemish upon an otherwise fine-appearing surface. For invisible patches, sheet asphalt is as good as any, perhaps the best, as cuts can be covered by it so as to be inconspicuous to a high degree. This sheet asphalt possesses, however, one great disadvantage in a town in which there is not enough of this kind of pavement to justify the establishment of a permanent asphalt plant within its limits. Cuts or trenches in such a case must wait for permanent repair until the traveling plant can make its annual or semi-annual visit to the town. This will often be found to be an important element in deciding smaller municipalities to use brick or block. A cut in a block pavement can be repaired by a good mechanic at any date desired, and need not wait for the arrival of outside parties. That many such cuts are carelessly or negligently repaired does not detract from the advantage. With proper care, brick or block laid upon a cushion coat of sand or fine gravel can be relaid so that the patch will not be out of shape with the rest of the street, nor unduly conspicuous. If the city employs incompetent workmen, or does not insist upon the competent man performing his task properly, the defect should be charged where it belongs and not to the pavement itself.

The heat-retaining power of a pavement is a feature which seems

to have been completely overlooked, and yet it is one which must exercise a great influence upon the comfort of the people living along the street, especially in southern latitudes. Some years ago the speaker made some experiments in Washington, which showed that macadam absorbed less heat than asphalt block, and that sheet asphalt absorbed more than any other kind of pavement. The amount of heat retained by it was sufficient to affect materially the temperature of the vicinity in hot weather. A city with a large number of asphalted streets must be hotter in summer than one with pavements of other materials. Maj. Beach.

The question is often raised as to whether or not a pavement excludes the moisture from the soil beneath it. The opinion formed from observations extending over several years in Washington was to the effect that in prolonged dry spells there was more moisture under the pavement than under the grass-covered places. The latter contain more water immediately after a heavy rain, but apparently the water, gradually diffusing itself through the ground, is retained under the pavement, while it evaporates from exposed places.

In Mr. Tillson's discussion, sheet asphalt, asphalt block and bitulithic are mentioned as the varieties of asphalt pavement. There is another variety which has been coming into use in some of our smaller towns, and, while too new to permit of definite decision as to its merits, seems to promise well as an inexpensive road covering in many localities. It is formed by raking or harrowing the road surface, then sprinkling it with fluid asphalt and afterward rolling it. The method is one which may be considered as half way between the oil sprinkling of roads and the bitulithic pavement. Tar has been used in some cases instead of asphalt and appears to have given good results.

After satisfactory results with one pavement, many engineers are inclined to recommend it, to the exclusion of others, but they should be careful to advise their clients according to their needs, or, at least, to show them the merits and disadvantages of the different materials and let them make their selection with a proper knowledge of the consequences.

E. M. T. Ryder, M. Am. Soc. C. E.—The speaker wishes to emphasize one point, namely, that of repairs. To a street railway man this is a serious question. The pavements of the present time are certainly not all that can be desired, and it is to be hoped that something will be discovered which will be an improvement. If joints in the track could be gotten rid of, the present trouble would be greatly diminished. The use of cast, thermit, or electrically-welded joints has been considerable, but is not universal by any means. It is necessary, for instance, in the case of electrically-welded joints, to have several thousand of them before it is economical to bring an electric welding plant to take care of them. It may as well be admitted that it will be necessary to take up joints in the track every once in a while, and, there- Mr. Ryder.

Mr. Ryder. fore, a pavement which would admit of doing so readily would be preferable. The pavement should be designed, not merely with reference to long life under favorable conditions, but with reference to its repair under conditions which will actually obtain. This is especially true in work where it is desired to use T-rails. There is a marked preference on the part of many city engineers for grooved rails, which undoubtedly have certain advantages, but it is an open question which type of rail is preferable, all things considered, and the decision must rest on local conditions.

In Connecticut, most of the steam and trolley roads are controlled by the same corporation, and in certain localities it is considered very desirable to run trains, single cars, or possibly trains of two cars, from one city to another, over the main-line tracks of the steam road, and then in on the city streets, in order to avoid transfers and to take people just where they want to go. This plan would afford a most convenient way for the people of the smaller towns to travel to the larger ones to transact business or to shop. In order to run trolley cars on the steam road, between the steam express trains, however, the same rate of speed is necessary for both. It would be unsafe to use the smaller flanges of the ordinary street-car wheels, or anything less than the M. C. B. standard dimensions, which practically means the use of T-rails for the street-car tracks. The only solution at the present time is to use granite dimension block along the rail, probably both inside and outside. In laying this granite block, the speaker prefers to use a tar or sand joint instead of a cement joint, because it is then more practicable to remove and replace the pavement for inspection and repairs.

A second and, of course, most important point in regard to pavements is the foundation under the track, as that often materially affects the life of the pavement. In towns where there is a good gravel foundation, no special preparation is necessary, but in some cities, where the soil is practically a brick clay, it is a very different matter. The company with which the speaker is connected has tried so-called standard styles of construction, including the use of concrete beams of great rigidity. With the concrete beams, however, the joints still have to be repaired, and these repairs have become very difficult.

The following brief description of a type of construction which has been suggested, is offered for criticism: First, make a simple rectangular concrete trough deep enough to take in the standard cross-tie and rail construction, allowing for a thin ballast of gravel under the ties and possibly a small tile drain. The supporting surface for the asphalt could be made comparatively thin, owing to the rigid, unyielding sub-foundation; possibly 3 in. of concrete laid just under the asphalt surface would be sufficient. It is a question whether any other type of construction can be found which would be preferable to the granite block for use along the rails, if a T-rail track is to be used.

When it comes to renewals, the salvage with this construction would be practically complete, as the foundation is as good as new, and it is only necessary to renew the rails and ties as in an ordinary street on a gravel foundation. Mr. Ryder.

The speaker would be glad to have an expression of opinion as to this type of construction, or suggestions of new types or modifications of existing types.

WILLIAM H. LAWTON, M. AM. SOC. C. E. (by letter).—In providing for any future pavement, the constantly increasing use of auto-vehicles must be considered. Some time, no doubt, in the distant future, they will entirely supplant horses as a motive power. At present, however, both must be considered, and, for the use of either, the future pavement should be indestructible; it should be hard and rigid, but not noisy; it should be smooth, but not slippery, either wet or dry; and it should be easily kept clean. Mr. Lawton.

The theoretical definition of a pavement is: An artificially improved roadbed, constructed of such durable material and presenting at all times such a smooth and firm surface as to facilitate street traffic of all kinds.

Street traffic is influenced greatly by the character and condition of the pavement, the width of the roadway, and whether encroached upon by street-car tracks.

All pavements are subjected to destructive action, principally by the combined forces of abrasion, crushing, and percussion, the latter, on account of the impact of horses' hoofs, being probably the most destructive. The weight of the loads and the grinding of the wheels are also large factors.

Its resistance to wear, or its durability, is the supreme test of every pavement; and its form or shape is of small account. The durability of a pavement depends on its ability to withstand compression and concussion, as to brittleness; and grinding and polishing, as to toughness. It is also subjected to loss and change by wear and friction.

Other important requisites are the imperviousness of the material and its ability to resist climatic conditions and changes. The pavement should also be sanitary, as to construction; and reasonable, as to cost and maintenance.

The relative durability of different pavements is measured by the amount of traffic each will stand before it becomes worn out. With regard to quality, there is at present no definite recognized standard except the general one derived from actual experience. Until some standard is adopted to determine the average life of different pavements, just conclusions can hardly be reached. Some standard, such as will measure the tons of traffic per square foot of area during a certain time, would seem to be most reasonable. The physical life

Mr. Lawton.

of a pavement, as regards repairs or renewal, should also be considered. When a pavement is laid upon a prepared foundation, and is monolithic in form, or is composed of broken stone with or without mixtures, the requisite repairs may often be made economically for a long period by incorporating new material. When pavements are of block form, there comes a time when it is poor economy to repair or relay them.

The annual expense of pavements has been represented by formulas derived by assuming that their length of life bears a certain relation to their original cost. More study and experience, however, are needed to render some of the variable quantities constant.

Most pavements can be classed under two forms, either block or monolithic; and in the Northern States a solid substructure or foundation is absolutely necessary for each. The prevailing foundation is concrete, from 6 to 8 in. deep, and composed of Portland cement and crushed stone. This foundation is the integral part of the pavement itself, and, under ordinary circumstances, will endure for ages. Upon it is laid the wearing surface selected, which, as it becomes worn, may be renewed or may be replaced by one of another kind. This wearing surface, or pavement proper, should depend principally upon the nature of the street traffic and the character of the street, but local interests or political influences often prevail. Many paving companies maintain a special agent, who not only extols the merits of his wares, but prejudices in his favor the proper authorities or the adjoining property holders. All real-estate owners recognize thoroughly the value of a good pavement, as not only is the street improved in appearance, but all property in that vicinity is enhanced in value.

There are many forms of block pavements, the principal ones being natural and artificial stone, brick, asphalt, glass, iron, cobble and wood.

The weakest part of each separate pavement is its edge, which first wears away—becomes chamfered in fact—leaving all the surface joints enlarged, and every block itself becomes rounded on top, and smooth and slippery. The concussion of horses' feet is the principal cause of this injury; and if only auto-vehicles were used, the deterioration would not occur as quickly nor extend as deep.

The blocks, also, are frequently of such size and shape, with such length of joints, that the pavement provides a poor foothold for horses.

The proper way to lay pavements is to embed them in a cushion of sand upon a heavy concrete foundation. Another important feature is the composition of the filler which is run in the joints.

Mr. Lawton.

The best filler is made up of fine gravel and a mixture composed of asphalt and coal-tar. This mixture renders the pavement less noisy, and, as it softens and expands in warm weather, it makes expansion strips unnecessary. Liquid cement or grout is sometimes used, but it tends to make the entire pavement absolutely rigid, with no chance for expansion. Cement joints are likely to be broken by heavy traffic, and the filler, not being plastic, never runs together again.

The best stone pavements are Belgian block, of trap rock, granite, and the hard sandstones.

As to the dimensions of these blocks, the opinions of engineers vary. Where horses are the motive power, the blocks should be as wide as the horse's foot. It should be long enough not to tilt, but not so short as to present a surface which is nearly all joints. As but few stone blocks are worn out by actual street traffic, they need be only deep enough to insure perfect stability. The surface dimensions of all pavements will remain a mooted question as long as street traffic continues to be by both horse and motor power.

No artificial stone pavement has yet been invented that will compare favorably with the natural stone blocks already described.

Among the monolithic pavements, the principal ones are those composed of Portland cement and of asphalts and bitumens. The wearing surface of the former consists of a layer, about 2 in. thick, of Portland cement concrete of about equal parts, mixed on the street, and incorporated in place upon the foundation already prepared. The surface is usually blocked off, by deep grooves, into squares of the desired size. This pavement is very slippery, especially in wet or frosty weather.

All the monolithic pavements deteriorate chiefly by injury from the impact of the calks of the horses' shoes. At present, most of the best forms of pavements mentioned, except macadam, appear to be equally serviceable for traffic by horses or auto-vehicles. Upon macadam the great dust raised by autos makes travel, not only very disagreeable, but even unsanitary. Also, the suction of the pneumatic tires, whether bound with chains or covered with leather bands full of iron rivets, destroys the binder, and ravels and scatters the crushed stone.

Sprinkling the surface with crude oil, or coating it with some preparation of tar, improves the wearing surface very much.

Owing to these defects, plain macadam paving should be considered as unlikely to be the pavement of the future.

From some form of cement or steel will probably be derived the future pavement. Experiments have been made with solid and hollow iron blocks, also hollow iron cylinders filled with concrete, and wooden blocks capped with steel plates; but none of these has yet fulfilled the requisites of a first-class pavement.

Mr. Lawton. Millions of dollars are being spent every year on city pavements, many of which are imperfect or wholly experimental. Is it not policy, then, for all municipal engineers to obtain, in a thorough and systematic manner, all the reliable data in relation to the wear of traffic in classified pavements?

Can the relative comparison of one pavement with another be made in any other way?

Is not this the only scientific method by which proper data may be obtained for guidance in the construction of the perfect future pavement?

Mr. Henry. PHILIP W. HENRY, ASSOC. M. AM. SOC. C. E. (by letter).—Pavements of the present may be divided, as to origin, into two classes, mineral and vegetable, and, as to surface, into two classes, sheet and block. As cheapness and durability are characteristics of mineral as against vegetable, and as noiselessness and ease of traction are characteristics of sheet as against block, it is evident that the pavement of the future will be of mineral origin, laid in an unbroken surface.

Existing pavements of mineral origin laid in sheet form are macadam and asphalt, both of which consist of mineral particles bound together with some sort of cementing material, differing only in kind, and in quality and size of particles. No doubt the pavement of the future will consist of mineral particles of the proper quality, proper size and proper cementing material. Just what will prove to be "proper" is a matter which will now be discussed. It is understood that in this discussion only the wearing surface will be considered, as it is assumed that in any case a suitable foundation will be adopted, whether such foundation consists of Portland cement concrete or other rigid material.

Quality of Mineral Particles.—In both macadam and asphalt pavements, while limestone and other soft stones have been used successfully in foundations, experience has proven that a hard tough stone must be used for the wearing surface. Thus trap rock, granite, or flinty limestone are preferable to the ordinary limestone, which, under traffic, is soon reduced to powder, necessitating constant repairs. For asphalt pavements, a siliceous sand is used almost exclusively, owing both to its abundance and to its wearing qualities. In some sections a shale or a calcareous sand has been used with satisfactory results where the particles are so small as to resist crushing. As a general proposition, however, it may be assumed that the pavement of the future will be composed of mineral particles of some hard, durable stone, such as quartz, trap rock, or granite.

Size of Mineral Particles.—While the quality of the mineral particles may be considered as definitely settled by past experience

in favor of a hard, durable stone, there is still a difference of opinion as to the proper size of these particles. It is generally conceded, however, that whether the largest particle is from 1 in. to 2 in. in diameter, as in macadam; or whether it is $\frac{1}{10}$ in. in diameter, as in asphalt pavements, the different particles, from large to small, must be graded in size so as to produce a minimum of voids. Thus, no matter what the size of the largest particle, there must be a sufficient number of fine particles to bring about that compactness and density so essential for durability. It is doubtful if this grading of particles has received much attention from those who lay ordinary macadam. This matter, however, has received most careful study and attention from those who lay that form of bituminous macadam called "bitulithic," which was introduced into America about five years ago, and since then has been laid in many cities throughout the United States and Canada. No doubt the success which this pavement has had over other bituminous macadams is largely due to the great care observed in proportioning the different sizes of mineral particles entering into its composition, beginning with 1 in. or $1\frac{1}{2}$ in. as the largest size, and continuing in definite proportions down to $\frac{1}{200}$ in. and less in diameter. By both theory and experience it has been found that certain definite proportions of different-sized grains are essential to produce the best results. Mr. Henry.

When the asphalt pavement, as known in America, was introduced, some thirty years ago, the necessity of proper sand grading was well appreciated, but in the effort to extend the business, demanding so much time and energy, the technical side was for a time neglected.

In the early days of the industry, pavements laid in different cities, with apparently the same proportions of sand and asphalt cement, gave very different results; and upon investigation it was shown that this difference was largely due to variation in the sand grading. At the present time it is well understood that, for the best results under heavy traffic, there should be a certain proportion of mineral particles $\frac{1}{10}$ in. in diameter, a certain proportion $\frac{1}{20}$ in. in diameter, and so on, down to impalpable powder. As this definite sand grading is not generally found in any one deposit, it is customary to mix sands from different deposits. As very few sands carry much 200-mesh material, it is customary to supply this very fine material in the shape of ground limestone or Portland cement.

In macadam the largest particle is from 1 in. to 2 in. in diameter, and in asphalt pavements the largest particle is $\frac{1}{10}$ in. in diameter. In the pavement of the future it is likely that both these gradings will be used, the coarser grading on streets of light traffic

Mr. Henry. where there is little danger of crushing, and the finer grading on streets of heavy traffic where compactness and uniformity are essential.

Kind of Cementing Material.—In ordinary macadam dependence is placed upon the stone or gravel itself for cementing material. Some limestones and gravels contain a certain amount of cementing material, and when properly watered and rolled the mineral particles will be tightly held together. Under heavy traffic, however, and especially since the introduction of the automobile, the finer particles in the macadam are soon separated and converted into dust. To overcome this difficulty, there have been introduced, for sprinkling macadam, various compounds, such as mixtures of water and calcium chloride, water and saponified oil, special crude oils, special products of coal tar, etc. All these, however, are useful only under the lightest form of traffic, and have as their chief recommendations the cheapness and ease with which they can be applied. This sprinkling is effectual only on the upper layer of particles, as the various solutions have comparatively little penetrating effect.

In order to get the best results it is necessary to coat each mineral particle with some kind of a cementing material, and to lay this mixture to a depth of from 1½ to 3 in., depending upon the amount of traffic. As cementing materials, both Portland cement and bituminous cement have been used. While Portland cement has been tried in various cities for a number of years, the results under heavy traffic have not been satisfactory, owing chiefly to the rigidity of the wearing surface, which would not stand the blows of traffic. Under light traffic, however, with proper attention to the grading of mineral particles, as above mentioned, there is, perhaps, a future for Portland cement owing to its cheapness as compared with bituminous cement.

For many years, both in America and in Europe, bituminous cement, in the form of coal-tar, has been used in the paving industry. Such pavements have been known as "tar macadam," "coal-tar pavements," "concrete pavements," etc. Owing, however, to its lack of uniformity and its volatility under the rays of the sun, coal-tar was practically abandoned upon the introduction of asphalt pavements. Within the past five years coal-tar has again been used extensively in bituminous macadam or bitulithic pavements. Whether a better and more uniform grade of coal-tar is now being produced than was the case thirty years ago is a question which time alone can determine. Owing to its cheapness, as compared with asphalt, coal-tar will doubtless be used in the future, as it is to-day, on streets of light traffic, but, owing to its susceptibility to changes of temperature and its more volatile nature, it is doubtful if it will be used for pavements subjected to heavy traffic.

Mr. Henry.

The word, "asphalt," first applied to pavements was the *"asphalte comprimé,"* known in America as "rock asphalt," laid in Paris in 1854. This was a limestone impregnated with bitumen, found in Switzerland, and later in France, Germany, Sicily, and other parts of Europe. To this day practically all the asphalt pavements of Europe are rock asphalt. On the other hand, in America there are very few rock asphalt pavements, due largely to the high cost of European rock asphalts, and to the inferior quality of those of America.

Practically all the asphalt pavements of America are composed of mineral particles cemented together with "asphalt cement." This mixture was devised by Edward J. De Smedt, a Belgian chemist, who came to America in 1861, and took out his first patents in 1870. The "asphalt cement" was composed of Trinidad asphalt, fluxed to the proper consistency with petroleum residuum; and the mineral particles were composed of sand and powdered carbonate of lime. This is still the combination used in many of the asphalt pavements of the present day. For twenty years or more Trinidad asphalt was used exclusively as the cementing material, but in 1893 an asphalt, occurring in Venezuela about 100 miles from the Trinidad deposit, known as Bermudez asphalt, was introduced, and has been used extensively ever since in the paving trade.

During the past six or eight years, in addition to natural asphalts, such as Trinidad, Bermudez, Cuban, Maracaibo, etc., there have been introduced bitumens manufactured from petroleums having an asphaltic base. These bitumens are known as "oil asphalts," "artificial asphalts," or "residual pitches." Just what part these particular products of petroleum will play in the asphalt pavements of the future, is a question which only time can determine.

There is no doubt that the natural asphalts are also products of petroleum, resulting from Nature's process of distillation. The crude petroleum has been forced to the surface by pressure from below, flowing down watercourses, as in Mexico; spreading over swamps, as in Venezuela; or filling the crater of an extinct mud volcano, as in Trinidad. Nature's distilling process, extending as it does for years, at the temperature of the sun, combined with the effect of the winds and rains, produces an asphalt well adapted for use in pavements which are exposed to all sorts of climatic conditions.

As there have been advances in the proper grading of mineral particles, so there have been advances in the proper consistency and proportions of asphalt cement. Various fluxes have been used for softening the natural asphalts, all of which are too hard in themselves. With some asphalts, results are apparently the same, whether the flux is made from the paraffine petroleums of the East, the semi-

Mr. Henry. asphaltic petroleums of Texas and Kansas, or the asphaltic petroleums of California. With other asphalts, the California fluxes give the best results; but, whatever the flux, it is generally considered that a certain consistency (known as penetration) of asphalt cement is desirable for certain conditions of traffic and climate, and well-recognized standards have been established.

There is still some difference of opinion as to the exact proportion of asphalt cement to be used in the paving mixture. This depends so much upon the kind of sand and dust, that no definite rule can be laid down. No doubt the future will bring about the solution of this problem along with many others.

Conclusion.—No doubt the development of the future will be in the line of various grades of macadam and asphalt, depending chiefly upon the kind and amount of traffic. That macadam is already the standard, for country roads and suburban districts all over the world, is well known, and no doubt an improved form of macadam will remain the standard of the future for this class of roadways. Like macadam, asphalt has become a standard form of pavement, especially for cities.

In 1876 Congress appointed a Commission, composed of the architect of the Capitol and two officers of the Engineer Corps, to select the best kind of pavement for Pennsylvania Avenue, Washington, to replace the wooden blocks laid only a few years previously. In all, there were more than forty proposals, representing all sorts of paving materials. Out of all these proposals, the Commission selected rock asphalt for one-third of the Avenue, and the "artificial siliceous" asphalt pavement (the ordinary asphalt pavement of to-day) for the other two-thirds. As up to that time there had been laid only 20 000 sq. yd. of the "artificial siliceous" asphalt pavement, the oldest less than five years, the Commission may be said to have had the courage of its convictions. That its judgment was correct is evident from the fact that in 1907 when the repaving of Pennsylvania Avenue again came up for decision, no other material for the entire Avenue was considered than that same variety of asphalt pavement, which, thirty-one years before, was selected for paving two-thirds of the Avenue.

Since that original selection, more than 60 000 000 sq. yd. of asphalt pavement have been laid in the United States and Canada, of which 18 000 000 sq. yd. have been laid in the past three years.

That other forms of pavement, such as stone blocks, brick, wood, etc., will remain in use, under conditions best adapted to these particular materials, is conceded, but the great development of the future will be along the line of macadam and asphalt, both of which have passed the experimental stage, and in both of which there are still greater possibilities.

B. BIENENFELD, M. AM. SOC. C. E. (by letter).—The rapid development and increasing use of the horseless carriage, or automobile, point to a largely extended and constantly increasing demand for smooth pavements. For many years, however, horses and other draft animals will be in evidence upon our streets, and provision must be made in the future, as at present, to facilitate the tractive efforts of these beasts of burden. On grades, and where heavy loads are to be hauled by animals, stone and brick, and possibly wood, will continue in use; but, as paving materials, these three substances possess many patent disadvantages, and their use will always be restricted to cases where the smoother and more ideal pavement is undesirable or unattainable. Mr. Bienenfeld.

The tendency toward the building of smooth, impervious, noiseless, dustless, monolithic, sheet-pavements is therefore ever on the increase. With the lights now before us, there is no reason to apprehend a falling off in that tendency. This will necessitate a large and reasonably cheap supply of asphalt. The asphalt fields of Europe, the West Indies, and South America, will continue to yield a reasonably large and cheap supply. Since their first exploitation in 1893, the asphalt-oil fields of California have given to the world an additional supply of asphalt; and it is possible that the asphalt-oil fields of Mexico may, in the future, be counted on as an additional source of supply.

As first made in California, the so-called oil-asphalt, produced by inexperienced manufacturers and laid by unskilled pavers, gave results which were far from acceptable; but, after fourteen years of experience in manufacture and laying, it may now be said that paving asphalt is being, and can be, produced from California oil, which is equal, if not superior, to any asphalt produced from any other source. It is, of course, the custom and habit of persons representing special interests still to condemn and to "damn with faint praise," this so-called "artificial" asphalt. But as "the proof of the pudding is the eating," so the proof of this asphalt is its wearing, and the constantly and rapidly increasing demand for it. It has found its way into successful paving use in nearly all the large cities of the United States and Canada.

During 1905, the latest year for which any statistical data are available, there were produced in California 50 000 short tons of oil-asphalt, which is the equivalent of 87 000 tons of refined Trinidad asphalt. In the fiscal year ending June 30th, 1905, the total asphalt imports into the United States from all foreign sources were 157 000 tons of crude, dried, "advanced," and rock asphalts. These crude materials contain from 20 to 70% of bitumen, whereas the California material contains more than 99% of bitumen. Allowing an average of 50% of bitumen in the imported asphalts,

Mr. Bienenfeld.

which is an excessive estimate, their total bitumen paving value would be 78 500 tons, as against the 50 000 tons of the California production. It is evident, therefore, that California, according to the latest data, is supplying about 40% of the asphalt requirements of the United States.

The oil production of California, which is to-day larger than that of any other State in the Union, is still increasing, through the discovery of new fields and the extension of the older ones; and there is every reason to believe that its asphalt production will be amply able to meet all demands that may arise in the next decade or two. In 1905, California produced 34 000 000 bbl. of oil, having an asphalt content of about 1 250 000 tons of pure, paving bitumen, of which only 50 000 tons, or 4%, were actually manufactured to meet current demands, the remaining possible 96% having been consumed as fuel.

The Republic of Mexico is at present producing asphaltic-oil, and may in the future enter the ranks as an important asphalt-producer, thus adding to the sources of asphalt supply. It is to be hoped that the producers of Mexican asphalt will profit by the early experiences and mistakes of the California manufacturers; and that they will allow no material to leave their refineries which is not in all respects acceptable and suitable for paving purposes. Care and technical skill are requisite; without these, the results are likely to be very unsatisfactory and possibly disastrous. For some time to come, however, all the Mexican oil production will probably be required to meet the fuel demands of that republic; and some years may elapse before Mexico can become an important asphalt producer. Its oil resources are at present largely undeveloped; and its asphalt possibilities are therefore as yet unknown.

The oil resources of other and more distant portions of the globe are still in reserve, unexploited. It is likely that some of them contain true asphaltic oil, similar to, if not identical with, the asphalt-oils of California. The asphalt resources of the West Indies and of South America are by no means exhausted. There is in sight, therefore, for future use, an ample supply of acceptable asphalt; and all the indications and tendencies of to-day make for a larger use of this paving material of the present in the construction of the pavements of the future.

Mr. Lewis.

NELSON P. LEWIS, M. AM. SOC. C. E.—This question, as the speaker understands it, is calculated to invite discussion or speculation as to whether or not the popular, successful pavement of the future will be one of those now generally in use, or whether something new will be evolved or developed. Almost every pavement is an evolution. An interesting thought has been brought out concerning another cheap bituminous pavement consisting of asphalt

laid on, or mixed with the ground. Here is a reversion to the dis- Mr. Lewis.
covery of the adaptability of asphalt for street pavements or road surfaces. The rock asphalts of Europe being hauled in carts for the purpose of making cistern linings, it was found that material which dropped from the carts, when mixed with the natural road material, formed a smooth, elastic and excellent roadway. This was probably the first asphalt pavement, and observation of the effect of the accidental deposit of asphalt on this road led to the construction of the first European rock asphalt pavement; and, after half a century, we are going back to this accidental type of roadway which brought about the development of the present asphalt pavement. This shows that, after all, bituminous pavements are the result of discovery and chance as well as of evolution.

In the speaker's judgment, the great trouble with the development of a new paving material is the disposition of the contractor or discoverer to exploit it before he understands it thoroughly. Reference has been made to wood pavements, beginning with the Nicolson block, made of the cheapest material and laid in the most indifferent way. Several cities in the United States had very unfortunate experiences with this pavement, one becoming practically bankrupt through laying large areas of these short-lived blocks. In America this gave wooden pavements a "black eye" for years. There is now a revival of wooden pavements, and there are evidences of the same disposition to exploit them by laying as many yards as possible at as good a price as can be secured through any influence that may be available. The greater the area laid, the more indifferent seems to be the workmanship, and the less care seems to be given to the selection of material, so that it will occasion little surprise if this material, which is capable of making an admirable pavement, is abused to such an extent that there will be a reaction against it in a very short time.

The adaptability of asphalt has been marvelous; it is found in successful use from Manitoba to the Tropics; in the severe frosts and storms of the North, and in the hot, moist atmosphere of the South. Under the conditions prevailing in the City of Mexico it seems to be admirable, and it is difficult to conceive of a pavement which could be more satisfactory than the asphalt pavements of that city. After all, the real value of a pavement is not determined wholly by its wearing qualities. Its appearance is a most important consideration, and at present there is no material that can be kept so neat and so immaculately clean as asphalt under an efficient system of street cleaning. This is admirably illustrated in the City of Mexico; the speaker cannot recall ever having seen asphalt pavements in better condition, or cleaner, than those laid on the principal streets. In walking to the Mineria, the speaker had a few scraps of paper in his hand, which he tried to dispose of; he had not the temerity, however, to throw them on

Mr. Lewis. the streets, which were perfectly clean, so he carried them into the building where he disposed of them. The City Government is to be congratulated upon its ability, not only to lay pavements of this kind well, but to keep them clean and maintain them very thoroughly. The officers in charge of street cleaning in northern cities might profitably visit the City of Mexico for an object lesson.

The selection of the kind of pavement is, after all, what might be called a question of luxury, as has already been said; how good a suit of clothes can a man afford to wear? Imagine the effect, on the rental value of an office building, of a clean pavement from which the dust will not be licked up and blown into the windows by every breath of wind, or of a pavement which is fairly noiseless. If distributed over that rent roll, the money required to renew such a pavement, if necessary, every three or four years, would be trifling. It is this luxury for which the people of London are willing to pay; the pavement in general use there is not permanent; it has to be renewed every few years, but it is clean and quiet; and the people are willing to submit to the inconvenience of its frequent renewal.

Block pavements are frequently urged and advertised, because they can be repaired so readily; but the argument is fallacious. There is no pavement that can be repaired as readily as modern artificial sheet asphalt pavement. If blocks are removed, they can never be laid as perfectly as they were originally, and when the true surface is gone, the blocks readily deteriorate. There is a disposition to relay the old blocks when they are not badly worn, but, with the sheet asphalt pavement, the material put in is new and has the same life as the original pavement. By the time the pavement is 10 or 15 years old, with the many frequent openings which are unfortunately made, a very large part of it is relatively new. This constant reopening of pavements is annoying and most exasperating, and it is to be hoped that some way will be found to avoid it. As far as the pavements of the future are concerned, the speaker can see nothing to indicate that there will be anything new. As Mr. Tillson has said, almost every material has been tried—glass, straw, iron, wood—almost every elementary substance, and every mixture which ingenuity or imagination can suggest. It is quite probable that the pavement of the future will be quite similar to the pavement of to-day, and the speaker knows of nothing which equals, in its universal adaptability, the modern sheet asphalt pavement.

Mr. North. EDWARD P. NORTH, M. AM. SOC. C. E. (by letter).—In any engineer's discussion of asphalt pavements, too much weight cannot be given to Mr. Richardson's closing sentence:

"The best contractors are always in favor of doing the best work compatible with the price they receive for it."

Mr. North.

Assuming the integrity of the foundation: the standard American sheet asphalt depends on the technical knowledge and faithfulness of the contractor's employees, combined with the quality of the contractor himself and the forcefulness of his favor toward good work, and the price received. All these uncertain elements, excepting the technical knowledge and faithfulness of the contractor's employees, can be controlled (theoretically) by the proper preparation and enforcement of specifications. But, if the engineer drawing such specifications has not had the advantage of some experience, both in works where the paving material is mixed and on the streets where it is under wear, his views may be based on theories rather than on a comparison of facts, and his specification be scholastic rather than scientific.

It is possible that some of the rigid, if not drastic, exclusions of materials which have proved satisfactory in the past, with minute directions as to manipulation, may result from theoretical considerations. All such specifications, so far as they govern work, are also an absolute bar to improvement either in materials or practice. But, in addition to any imperfections in the specifications, there is the ever-present practical impossibility of efficient inspection under the municipal laws obtaining in the United States. The inspectors are selected by civil service examiners who generally know nothing of asphalt, and the examinations show nothing of either the integrity or ability of the examinee to secure good work. It is possible that neither the appointee of a boss, nor the Civil Service Servant can be removed for incompetence without friction and sometimes potential interference.

All the above-mentioned dangers to good asphalt paving were minimized, though not entirely eliminated, by the specifications devised for the City of New York by Stevenson Towle, M. Am. Soc. C. E. In substance, they allowed considerable latitude in the choice of materials; called for 15 years' maintenance, with a bond satisfactory to the Comptroller; paid 70% of the contract price on the acceptance of the work, retaining 30 per cent. All of this 30% was held until the sixth year, when and thereafter 3% was to be paid yearly until the expiration of the term of maintenance. Until the winter of 1897-98 these specifications, throwing the responsibility for the best work on the contractor, gave the old City of New York, now the Borough of Manhattan, smoother and better pavements than the *asphalte comprimé* of London, and quite as good as those of Paris.

Since the above-mentioned date, the system has broken down. Neither the various Mayors, Borough Presidents, Comptrollers, nor Commissioners of Accounts, a commission said to have all the authority necessary to protect the City against peculation and fraud, have apparently known through this period of nearly ten years that the contract for maintenance was being disregarded. And these city officials, some of whom were frankly bad and others who were said to

Mr. North. be better, have placidly paid, or seen paid without protest, approximately $150 000 a year for the maintenance of pavements which everyone knew were not being maintained.

If this neglect is thought typical of American municipalities, the short-term contracts with rigid specifications, as at present advocated, are undoubtedly advisable, but if dependence can be placed on the loyalty to the interests of a municipality by its inhabitants, the system of long-term contracts with suspended payments will insure the best work without strict compatibility with the price received.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 1059.

WATER SUPPLY.

An Informal Discussion at the Annual Convention, July 10th, 1907.

SUBJECT FOR DISCUSSION:

"Is it better policy to purchase and control water-sheds, thereby preventing the pollution of impounding reservoir supplies, or to suffer a certain amount of pollution of such supplies, relying upon filtration to correct the effects thereof?"

BY MESSRS. GEORGE W. FULLER, F. HERBERT SNOW, D. D. CLARKE, GEORGE A. SOPER, WILLIAM J. BALDWIN, GARDNER S. WILLIAMS, J. T. NOBLE ANDERSON, WILLIAM B. FULLER, J. WALDO SMITH, PABLO SOLÍS AND OCTAVIO GUZMÁN, S. BENT RUSSELL, M. R. SHERRERD, F. W. BLACKFORD, GEORGE A. JOHNSON, AND GEORGE W. FULLER.

Mr. G. W. Fuller.

GEORGE W. FULLER, M. AM. SOC. C. E. (by letter).—Various local conditions require consideration, in deciding this question. This is particularly true as to the quality of water which it is desired to obtain, which quality must meet, or ought to meet, certain rational requirements of local or State health authorities.

It is undoubtedly true that water supplies which were believed to be good 10 years ago are not in all cases now regarded as satisfactory in quality. As to water supplies of good quality to-day, there are strong reasons for believing that some of them will not necessarily be thus regarded a few years hence. We are in a transitional stage as regards the quality of public water supplies. Advanced positions taken by various health authorities and by public opinion, without doubt, now make it urgent for the engineer to be keenly alive to sanitary requirements, in order that public water supply projects may keep abreast of the times.

Mr. G. W. Fuller.

The basic thoughts or premises should first be stated, in order to facilitate discussion, and, as they appear to the writer, they may be outlined briefly as follows:

1.—It is essential that the water from impounding reservoirs, or from any other source, shall be substantially free from disease-producing germs.

2.—A satisfactory public water supply should be of good appearance and free from objectionable quantities of mud, turbidity, and vegetable stain.

3.—A satisfactory water supply should be free from objectionable tastes and odors.

4.—Impounding reservoirs are of much assistance in the purification of water through natural agencies, as regards the removal of bacteria, turbidity, and peaty color. Their agencies are not infallible, and their efficiency varies much with different conditions as to rainfall, temperature, climate, storage period, and other physical factors.

5.—The elimination of pollution through the purchase of land is highly desirable, but in many places is prohibited by its cost. Sanitary control through inspection is helpful, but difficult of accomplishment under existing laws and customs.

6.—A well-designed and well-operated filter plant, with all its various appurtenances, such as efficient aeration, sedimentation, and coagulation, will allow a satisfactory water to be obtained from practically any impounded-water supply, although, within certain limits, the better the quality of the unfiltered water, the better and cheaper will be the filtered water.

Protection by Impounding Reservoirs from Disease Germs.—Where a reservoir has a capacity equal to the volume removed from it during a period of many weeks or months, it is obvious, from our present knowledge, that a large proportion of the disease germs will disappear from the flowing water. These germs have weight, and subside in fairly quiet water. According to laboratory experiences, disease germs live much longer in reservoir water than in sewage or highly polluted waters.

In all waters there is quite a rapid dying out of disease germs during the first week or so, and, with present laboratory methods, it is usually very difficult to detect the presence of any of these germs after they have been in water for two or three weeks. This does not prove, however, that all the germs have died out. On the contrary, the evidence seems to be quite strong that some of the more hardy ones, perhaps only a fraction of 1% of the total number, will survive for a long period, presumably months and years. Otherwise, it seems to be impossible to explain the continuance of water-borne diseases in certain districts.

Mr. G. W. Fuller.

While our knowledge of the passage of disease germs through large impounding reservoirs is by no means precise, our information is such that we cannot ignore certain physical conditions limiting the efficiency of the natural agencies there at work. Among such are the stirring up of the bottom layers of deep reservoirs, due to vertical circulation during each spring and autumn, and the effect of stratification and of various currents, which, under some conditions, no doubt bring the entering water to the outlet in an abnormally short period of time.

It is futile to go into this subject in great detail here, but it is to be pointed out clearly that because a reservoir may hold 100 days' supply it by no means follows that the numbers of disease germs passing through the outlet are always as low as would result from the stay of these germs for 100 days in the reservoir water. Obviously, the protection from impounding reservoirs increases with the period of effective storage.

There are other factors also to be considered. Among them may be mentioned heavy rains, which sometimes wash into the impounding reservoir the accumulated filth of weeks and months, and also that such pollution may at intervals occur when the reservoirs are drawn down quite low, and when the opportunity for purification by natural agencies within the reservoirs is abnormally small.

Let us next turn to the question of how much pollution may suffice for impounding reservoir waters to produce disease among the consumers. The evidence is quite striking that the dejecta of one or two persons suffering from typhoid fever or other serious intestinal disorders may pollute surprisingly large volumes of water. This is particularly true when such dejecta accumulate during the winter months and are suddenly washed into the stream or reservoir when there is a general thaw. Experiences at Plymouth, Pa., New Haven, Conn., York, Pa., Scranton, Pa., and other places, show that the danger from a scattered population on the water-shed may at intervals be far greater than was considered possible a few years ago.

Notwithstanding its shortcomings, the impounding reservoir has been and continues to be a tower of strength in municipal sanitation in many places. This becomes quickly apparent upon comparing the typhoid death rate (about 20 per annum per 100 000 population) in New York, Newark, Jersey City, etc., which is as low as in numerous other cities more or less similarly situated, and having a public water supply either from underground sources or of filtered river water.

Filtration of upland water from sparsely populated areas, however, produces still lower death rates in some cases, as shown by the experiences of Paterson, N. J.

Mr. G. W. Fuller.

Mortality statistics as to typhoid fever and other water-borne diseases are frequently only a very rough measure of the quality of a public water supply. This is particularly true of waters which are of fairly good character. Pollution may exist at times without its presence being recognized. This is probably true at intervals in the case of the Croton supply. Early in the spring of 1907 there was a small outbreak of typhoid fever in New York City, stated by Health Commissioner Darlington to have been caused by Croton water.

One of the weaknesses of the impounding reservoir is that its inefficiency cannot always be detected or corrected in time to avert danger. This accentuates the importance of more attention being given to one or both of the procedures advanced in this theme for discussion, namely: the prevention of pollution on the water-shed, or its correction by filtration.

Protection by Impounding Reservoirs from Unsatisfactory Physical Condition of Water.—Sedimentation in large impounding reservoirs usually produces sufficient clarification, and it is only at times of exceptionally heavy rainfall that the water is muddy when delivered to the consumers. As compared with the waters in large rivers, especially in the South and West, the impounded supply is quite satisfactory in this regard.

The remaining turbidity in the water drawn from an impounding reservoir, however, is usually due to very fine particles, dark in color, which produce what is called a "dirty appearance" in the water; this is frequently complained of much more than would be expected by the users of muddy river waters.

In the case of waters drawn from drainage areas containing more or less swampy tracts, the impounding of the water does much to bleach it by the aid of sunlight, and to lessen materially the brownish tint due to organic matter extracted from leaves and other vegetation.

It is with regard to tastes and odors that the most disagreeable physical conditions are likely to be encountered in the water of an impounding reservoir. These may be due to the decomposition of organic matter on the bottom of deep reservoirs, or to the growth and disintegration of various small forms of vegetable and plant life which make their appearance in reservoir waters in a more or less irregular fashion and in a manner not now fully understood.

The stripping of the bottom and sides of reservoirs does much in the early life of a reservoir to reduce the likelihood of bad tastes and odors due to vegetable growths. It is not an absolute safeguard against these troubles, however, and it is to be further pointed out that the effect of stripping is not wholly permanent in its nature. In fact, present evidence indicates that soil washings,

in course of time, perhaps 10 or 20 years, will largely obliterate the effect of stripping. Mr. G. W. Fuller.

When it is considered that a public water supply ought to be thoroughly "clean," it can hardly be maintained, in the light of modern requirements, that an ordinary impounding reservoir can give uniformly a thoroughly satisfactory supply in the absence of filtration, or of aeration, or of both.

It is true that swamps may be drained and various other expedients resorted to, but it is not ordinarily possible, in the opinion of the writer, fully to meet modern requirements as to the physical conditions of water through the agencies which act in the ordinary impounding reservoir. There are, of course, exceptions to this rule, and some exceptions will no doubt continue for a long time to come. The experiences and conclusions, however, at several places where bad tastes and odors prevail, show that, independent of pollution upon a water-shed, filtration is highly desirable and well worth its cost. The fact that trouble from microscopic growths has not yet been experienced in any given impounded supply is no proof that difficulties will not arise in the future.

Protection by Filtration.—For the most part, filtration has been practiced in America in connection with muddy and polluted river waters. For impounded water supplies, sand filters have been used successfully for two of the gravity supplies at Reading, Pa., the Ludlow supply at Springfield, Mass., and have been introduced recently at Bar Harbor, Me. Impounded water supplies have been treated satisfactorily by mechanical filtration at a number of places, among which may be mentioned Lexington, Ky., Middletown, N. Y., Norfolk, Va., and Charleston, S. C.

Available evidence shows clearly that by filtration a water can be made thoroughly satisfactory as to freedom from disease germs and objectionable tastes and odors, as well as of satisfactory appearance. Modern filter plants, both in their construction and their operation, require close adjustment to existing local conditions, and are bound to vary somewhat in their detailed arrangements. Filtration, in this discussion, is viewed in its broadest light, and with it is included efficient aeration and the use of coagulants when necessary.

As to the cost of filtration, that of an installation, complete, with all appurtenances, usually ranges from $15 000 to $30 000 per million gallons daily capacity. Usually, the figures for impounded reservoir waters would be nearer the lower figure, although there are special conditions in some places which require the higher figure to be exceeded. The cost of operating filters of moderate size usually ranges from $1.50 to $3.50 per million gallons for reservoir water, exclusive of pumping. Combining the operating expenses and capi-

Mr. G. W. Fuller.

tal charges, taken at 5% per annum, there is obtained, for fairly representative conditions, the sum of $7 per million gallons. On the assumption that 100 gal. of water are used daily by each inhabitant, this would mean a total annual expense of about 25 cents for each inhabitant.

Filters ordinarily remove from 97 to 99.5% of the bacteria in the applied water. Where bacteria have been removed to a large extent by sedimentation in impounding reservoirs, the numbers are sometimes so low in the unfiltered water that the above percentages are not obtained. The residual number of bacteria, however, would be very low, and the water would be fully equivalent in hygienic quality to the above stated percentages of removal as we understand them in connection with practical operations in the filtration of river waters.

Filters are not germ-proof, and none of their warmest advocates claims them to be. Their efficiency has proved a tower of strength in municipal sanitation in the past—even greater than that of the impounding reservoir—notwithstanding the fact that a small fraction of the applied bacteria passed through the filters and appeared in the water supplied to the consumers.

Obviously, the more polluted the unfiltered water is, the less safe will be the filtered water. The logical question in connection with this discussion is this: To what extent can water-sheds be purchased and controlled, with the result that the water is made of thoroughly safe quality, and under conditions where the investment on the water-shed is preferable to an investment at a filter plant?

Two fundamental things are to be borne in mind: First, that, all things being equal, a pure water is better than a purified water; and, second, that by adjusting the rates of filtration, by double filtration, and by the use of germicides, it is perfectly practicable for filtration experts to deliver to the consumer almost any grade of filtered water that is wished, provided the purification works are built and operated strictly in accordance with the best available knowledge of sanitary science.

This brings us to the question of relative safety, and the reliability of human efforts, both as regards the control of water-sheds and faithfulness and efficiency in the operation of filtration works.

Comparative Cost and Reliability of Preventing Pollution on the Water-shed and of Removing Pollution by Filtration.—Data are available to permit, for any specific case, a fairly close estimate of the comparative relations of these factors. It is difficult, however, to generalize on this question, owing to the varying local conditions, and they will be treated in this discussion in an illustrative way.

Mr. G. W. Fuller.

The total cost of efficient filtration may be taken at an average figure of about $7 per million gallons per day, including interest, sinking fund, repairs, replacements, and operating expenses. This is equal to about $2 500 per annum, corresponding to about $50 000 as the sum required for the maintenance of filtration works of a capacity of 1 000 000 gal. daily in perpetuity.

A water-shed area corresponding to a dry-weather yield of 1 000 000 gal., of course, depends much upon the rainfall, the evaporation, the relative capacity and water surface of the impounding reservoirs, etc. For purposes of illustration, let us assume that 1 000 acres of water-shed area are required for the development of 1 000 000 gal. of water daily. This means that, for the purchase and control of water-sheds, there would be available about $50 per acre, in order to have the cost of such treatment approximate that of filtration.

Where land is cheap, the water-shed area small, and the reservoir capacity large, there is merit in purchasing the entire land in some cases. For projects of moderate size, the writer believes that the likelihood of such purchase doing away permanently with the needs for filtration is too remote to be considered seriously.

Based on the above assumptions as to cost, it is seen that, for large water-sheds, where there are numerous villages, fertile farming lands, and desirable locations for country residences, it is hardly feasible to purchase the entire water-shed for anything like the sum which would permit efficient filtration to be maintained for all time.

If the costs were equal, and only one procedure were to be adopted, it is the writer's opinion that filtration would be preferable, not only on hygienic grounds, but for the sake of assuring a water supply of suitable physical character at all times.

In this it is to be borne in mind that filters have been installed recently at Bar Harbor, Me., notwithstanding the fact that there are no regular residents upon the water-shed, although it is stated that there are some three or four summer camps. Bad tastes and odors required the adoption of filtration.

As to the purchase of those lands upon which there are sources of objectionable pollution, and the control of the remainder by sanitary inspection, the writer is aware that this can be done with much benefit upon small water-sheds, but that it is frequently difficult to secure satisfactory results on large water-sheds by purchase and control.

The City of New York has undoubtedly benefited to a considerable degree by the elimination of objectionable sources of pollution upon the Croton water-shed, through the purchase of large numbers of small tracts of land, especially along the watercourses. However, the writer is keenly appreciative of the limitations to

Mr. G. W. Fuller.

these methods as applied to the Croton water-shed of 360 sq. miles of drainage area.

Italian settlements on low, wet land may be purchased to-day, but they will certainly spring up again to-morrow in almost equally objectionable locations.

Sanitary inspectors may provide thoroughly sanitary arrangements for the removal of fecal discharges, but of what avail are they with a population whose habits are such that they will not use these sanitary conveniences, and whose lack of education is such that they persistently and regularly resist efforts to protect both themselves and the users of the water from the drainage area upon which they are resident?

Several things must be corrected before the purchase and control of water-sheds for large projects can approach the efficiency of filtration when conducted with moderate intelligence and skill as practiced in numerous places to-day.

A campaign of education is required, not only for the Italian on the Croton water-shed, but for many others whose views would no doubt respond to educational efforts to correct their ignorance and indifference. Such corrections will assume practical shape largely in proportion to the vigor with which wise sanitary laws are carried out promptly, and with a firm hand, to administer proper punishment in an equitable manner, and about which there can be no dispute as to the division of responsibility between State departments of health, municipal departments of health, and municipal water departments.

The history of sanitary control of water-sheds in America, in the opinion of the writer, shows that it does not offer nearly the protection to the water consumer that is afforded by modern filtration. Sanitary control has been of great help in numerous places, but there are other instances where laws are inefficient, residents on the water-shed careless and indifferent, and where efficient results are more or less confined to the period when the inspector is actually on the premises. Better laws, better and larger organizations for sanitary inspection, and a better understanding by the residents of what is required can no doubt bring about improved results in the future.

While the writer is not in favor of sanitary control to the exclusion of filtration, he is clearly of the opinion that more stringent sanitary laws than now exist in most places should provide for the removal of all gross pollution, and that simple, practical methods should be gradually introduced to eliminate uniformly the lesser pollutions in the order of their importance.

Mention should be made of the reliability of filtration as actually carried out in practice. A decade ago many filter plants were such

Mr. G. W. Fuller.

in name only. The last few years have seen a marked change for the better. Practically all the larger filters are now well designed, and are arranged so that they can be controlled readily by intelligent, faithful attendants. Most important of all is the almost universal custom of installing laboratories at the filters, and of placing the operations under constant technical supervision. In no branch of sanitary engineering has more progress been made lately than in the efficient management of water filters for the larger cities.

In the future, such management will certainly continue, as it will doubtless be obligatory to furnish frequent and full records of the results of operation to supervising State authorities.

Compared with sanitary inspection, scattered at different points over a water-shed, and in the hands of various individuals, it appears to the writer that there is far less likelihood of inefficiency in the operation of filters, centrally located, with the management under a single roof, and in the hands of a single person.

European Tendencies.—In Germany, where for a dozen years filtration has been practically compulsory for surface-water supplies, there are only a very few important cities which derive their water from impounding reservoirs. In Great Britain, however, there are many such supplies, and it is significant that the majority of the engineers having charge of new impounded-water supplies have provided not only for filtration, but also for the purchase of the water-shed, where this can be done at moderate cost. Fortunately, such purchase has been practicable, and has been carried out, at the new water-works of Liverpool, Birmingham, Swansea, Cardiff, Manchester, Edinburgh, etc. Land on these drainage areas is comparatively inexpensive, and the writer thinks that there is no room for doubt about the wisdom of eliminating at nominal cost all sources of pollution from a resident population.

There are various reasons why it is usually difficult in America both to filter and to purchase or control the water-shed, notwithstanding the desirability of doing both, from a theoretical standpoint. It is even necessary in some cases to install filters for polluted streams, when impounded-water supplies could be availed of as far as physical conditions are concerned. Under such circumstances, it is the writer's practice to arrange the filters so as to serve for the upland supply when financial and other conditions will permit its adoption.

Conclusions.—1.—The purchase and control of water-sheds is a less efficient procedure than filtration, as regards both the sanitary character of the water and its physical condition as to appearance, tastes, and odors.

2.—For projects of much magnitude, filtration is generally the cheaper.

Mr. G. W. Fuller.

3.—Sanitary control of a water-shed is of much assistance, but it is usually a difficult matter to manage well at all times for large water-sheds, under existing conditions as to sanitary laws, and as to the views and responsibility of rural population. Better laws are generally needed.

4.—The purification of water due to natural agencies in a large impounding reservoir makes its filtration easier, cheaper, and more effective than in the case of the ordinary river water. Purification plants can be installed to insure almost any degree of efficient purification that is desired.

5.—Notwithstanding the efficiency of filtration at reasonable cost, there is a growing tendency in America to provide as good sanitary conditions on a water-shed as practicable. As a rule, it is not believed that the purchase of the entire water-shed is advisable, on the ground that the money involved can be expended better in other ways.

Mr. Snow.

F. HERBERT SNOW, M. AM. SOC. C. E. (by letter).—The sanitary conservation of public water supplies is a subject which forced itself upon the attention of the General Assembly of the State of Pennsylvania in 1905, resulting in the enactment of a law entitled "An act to preserve the purity of the waters of the State, for the protection of the public health," under which the Commissioner of Health is administering that branch of State medicine relative to public water-works and sewerage in a way to prevent ultimately a recurrence of epidemics of a water-borne character.

During the winter of 1906-07 three notable epidemics occurred in Pennsylvania, namely, at Scranton, Warren and Kittanning. Prior to this time the State had furnished the classic typhoid fever epidemic at Plymouth, and later at Butler. The facts in connection with these cases and the lessons they have taught should be clearly in the mind of the expert called upon to advise with respect to the amount of sewage pollution permissible on a water-shed.

The Scranton typhoid scourge came from an unfiltered surface supply of water derived from a mountain water-shed on which were located three villages and two railroads. The disease struck the town suddenly, lasted about 6 weeks, totaled more than 1 000 cases and rolled up a record of more than 100 deaths. No infection could be found on the water-shed. The poisoning of the water must have been accidental, but, nevertheless, it was certain. Typhoid bacilli were discovered in one of the reservoirs.

The lower basin, from which the gravity supply mains extend to the town, has a capacity of about 100 000 000 gal. The main impounding reservoir is 5 miles above, and has a capacity of more than 1 000 000 000 gal. A short distance above is a third impounding reservoir having a capacity of about 400 000 000 gal. Extending for

several miles along and across the main stream, and by the shores of the three reservoirs, are the two railroads. Droppings from the passenger coach closets might easily have been washed from the roadbed into the city supply. Mr. Snow.

Although these and numerous other permanent menaces had existed on the water-shed for years, Scranton had never before been afflicted by a water-borne disease. Whatever pathogenic pollution had reached the reservoirs was either effectually destroyed by natural agencies or rendered impotent. Retention in the impounding basins must have been one of the formidable barriers against the entry of infection into the water pipe system. No doubt is entertained about such infection having been present in the city supplies prior to the epidemic. The topographical evidences are conclusive.

It was not due to improper operation of the water-works plant, or to any apparatus or appurtenance thereof, that the flood gates of disease were let down in this instance. Simply this: there came a day when all the natural barriers proved of no avail. Without any change in the physical conditions on the water-shed, and without warning, Nature's safeguards went down. What had proven for thirty years or more to have been an adequate plant for furnishing a pure and wholesome supply suddenly became inadequate, thereby transmitting disease and death to the community. The origin of the infection will probably never be known.

An intelligent mind cannot reflect on the history of this catastrophe without being impressed by the lesson it teaches. Barriers against water-borne diseases, to be effective under all conditions, should be capable of manipulation and control. The water company had purchased several hundred acres along the streams and around the reservoirs, and had attempted to control the water-shed by a system of patrols constantly maintained. Many nuisances were thus abated and some prosecutions made to prevent the pollution of the impounding reservoir supplies. During the epidemic, the State assumed control of the water-shed, and put in force rigid rules and regulations for the sanitary protection of the waters thereof. There are scores of public water-works systems to-day where the sources of supply are as likely to be infected, accidentally or otherwise, as the Scranton source, and even more so. Many human lives are bound to be sacrificed before adequate and universal preventive measures shall have been adopted.

At Warren, beginning on December 8th, 1906, an epidemic of gastro-enteritis raged for four days, during which 1 200 patients called for medical attention, and about 600 more were reported bedridden, but did not receive professional care.

This borough takes its water supply from driven wells, in the

Mr. Snow. town, on the banks of the Allegheny River, the waters of which are polluted by sewage. Investigations by the Department of Health pointed conclusively to the water as the medium of transmission of the poison. From the nature of the ailment, it was premised that the infection must have been in concentrated form.

The cause was discovered to have been the sudden rising of the river and the back-flooding of sewage through a drain pipe and ultimately through loosened joints of the piping of the driven-well system and into the town's supply. Attention to the sewer, which was in a leaky condition, and to the joints and packing of the suction pipes, which joints and packing had been loosened by alterations in the main suction pipe, would have prevented trouble. Too much care cannot be exercised with respect to the condition of the apparatus by which ground-water is obtained for a public supply, when such apparatus is located on the banks of a sewage-polluted river in which freshet heights inundate the surface of the land upon which the wells are located. The danger exists if there be sewage in the river, or a conduit, or a receptacle, or anywhere whence it may be transmitted to the drinking water. The lesson taught by this experience is that a ground-water supply, regularly proven by tests to be of superior quality, may suddenly become contaminated from a known remote source, and be prejudicial to the public health. The remedy, therefore, if it be practicable to do so, is to remove the remote danger, in other words, to pay attention to the proper disposal of sewage.

The sewers of Warren discharge into the Allegheny River about 90 miles above the borough of Kittanning, in which the source of public supply is the same river. The water was filtered, and was supposed to be purified. However, expert investigation revealed the fact that the plant had not been operated intelligently, nor was it equipped to render satisfactory treatment of the water possible. Yet the public was led to believe that the system of purification as installed and operated was a sufficient safeguard.

Several days after the Warren outbreak a similar epidemic of gastro-enteritis broke out in Kittanning. It lasted more than 8 weeks and numbered in that time upwards of 4 000 cases. The physicians of the borough reported that the vast majority of citizens were afflicted and received medical attention.

Pathogenic pollution from many municipalities on the banks of the river above Kittanning may arrive at the water-works intake at the latter place in condition capable of causing disease. The Warren infection might have been transmitted to Kittanning and have produced the epidemic there. The coincidence of outbreak in the two places attracts attention, although a local source of infection is more probable. But, besides the bowel trouble, there were

more than 100 cases of typhoid fever in Kittanning. This infection is believed to have been introduced through the medium of the river. Mr. Snow.

This case demonstrates how utterly inefficient may be the effort of those who, misinformed as to the essentials of water filtration, try to safeguard human life. Here was a water purification plant, installed for the purpose of preventing a recurrence of typhoid fever, totally inadequate to perform the service expected of it by the public, but, nevertheless, serving to foster a false sense of security.

In such a case, where the source is always dangerous, the interests of the public health demand competent design and skilled supervision of the construction of the purification works at first, and thereafter, capable management of its operation, and efficient maintenance.

The business of furnishing drinking water to the public is bound to be regulated by laws, passed to be enforced by central administrative power. The courts are beginning to give a different meaning to the words, so frequently appearing in water-works franchises, "pure and wholesome supply." Practical methods have been evolved whereby it is possible now to render water purer than it may be possible to obtain it in a raw state. The consumer is entitled to the kind of article he purchases, that is, a "pure and wholesome supply," and a Pennsylvania court has recently held that primitive methods of water purification, whereby a slightly better water than exists in the stream is furnished to the consumer, is not sufficient as long as that water is prejudicial to the public health.

The Butler epidemic, of the winter of 1903-04, affords a different lesson. At that place the public water supply was adequately filtered by a plant properly designed and operated. The source was from a rolling farming district, yielding turbid waters at times, and subject to possible sewage pollution. The filters had been installed to eliminate all danger and produce a clear water. This they did. The plant was one of the advertised assets of the borough; therefore, when the town woke up one day to the presence of considerable typhoid fever in the community, no one, not excepting even the newspapers (so alert usually to scrutinize critically a public utility plant), seriously suspected the water supply; and, remarkable to relate, not until 1 000 people were stricken and in bed was State aid called for and the true origin of the scourge discovered. The disease did not come in a day, but was cumulative.

The filter plant had undergone repairs just prior to the inception of the epidemic. During this period of repairs—several days—crude creek water was introduced into the street mains. There were several cases of typhoid fever on the water-shed, in a dwelling on the banks of the stream. The dejecta from the patients were

Mr. Snow. thrown out on the surface of the ground, where nothing prevented the poison from being washed into the creek, and hence within a few hours it arrived in the homes of the unsuspecting, confiding and wholly helpless water consumers.

Had the local or State health authorities been made aware of the first outbreak of the disease, or of the original cases on the water-shed, the history of the Butler epidemic might never have been written. But, as it was, 1348 cases were recorded and 111 deaths.

This catastrophe produced one good result, namely, the establishment of the State Bureau of Vital Statistics. At present, physicians are required to send in morbidity reports. A repetition of the Butler case is not possible. That is, a community cannot be gradually attacked by typhoid. The onslaught must be sudden and sufficiently intense to cause the harm at once, because the officers of the local and State health departments are ever on the lookout, and report to the central office the first appearance of any infectious disease.

However, systems of reports can only apprise of danger. The poison may be disseminated through a water-works system in a few hours, and the first infection may take place before knowledge of it is possible. To the initial infection at Scranton should be attributed about all the cases there. Had the disease escaped detection in its early inception, the epidemic would undoubtedly have far exceeded the proportions it assumed, and thus might easily have gone down in history as the most appalling typhoid scourge on record. Thus the early discovery and the shutting off of the infected water in this one instance alone probably saved to the community, besides human life, a money consideration greater than the amount expended annually on records by the State and city Departments of Health.

In another instance, Johnstown, by the prompt report of typhoid on the water-shed, was saved from an epidemic under circumstances otherwise assuring a sweeping disaster. The poison was intercepted and killed, and not a case occurred in the town. Unquestionably, the morbidity reports instituted by the Commissioner of Health will repay to the public the cost of maintenance of the Bureau, and this work has been made necessary in following out the policy of preventing pollution of impounding reservoir and other surface supplies, as far as it is practicable to do so.

The Butler lesson teaches that an efficient filter plant is not an absolute safeguard; that as long as there is poison in the source, the danger of an explosion of disease of a water-borne character exists.

The Plymouth epidemic of 1885 attracted the attention of the world's sanitarians. The case was unique at that time. A town,

nestled at the foot of a mountain, furnished with a pure mountain water coming from an uninhabited water-shed—excepting one dwelling—was suddenly the scene of the terrible scourge of typhoid which numbered 1 104 cases and 114 deaths. The discovery of the origin was, for that day, a masterful work of detection, clear-cut and uncontroverted. Mr. Snow.

The owner of the single dwelling-house on the water-shed went to Philadelphia, contracted typhoid, came home, and was attended by a physician and nurse who did not know that the little stream at the foot of the bluff, on the edge of which stood the cottage, was the source of Plymouth's drinking water, and so they threw the poisonous stools out on the ground, where the dejecta remained frozen during the cold winter months; but in April, when the thaw and warm rains came, and the ice and snow all disappeared, the typhoid poison, along with everything else, was carried into the creek, and thence some of it found its way through the water pipes and into the town.

This was not the only supply to Plymouth. The Susquehanna River, receiving the sewage from Wilkes-Barre, 3 miles above, was used occasionally for parts of the town. This source was thought to be dangerous because it contained sewage, and suspicion had been directed to it for a long time. The public persisted in attributing the epidemic to the Susquehanna River water, until the proof of the true origin became conclusive. Although Wilkes-Barre's sewage constituted an immediate and always present menace, yet the citizens of Plymouth had not, prior to 1885, suffered from typhoid more than had other communities in the vicinity having sources of water supply which were not from the river. Contrary to the expectations of those interested in this case, came the proof that a constantly sewage-polluted river supply used by people who know the danger in the water, may not be as prejudicial to public health as a mountain supply free from pollution and having but one apparently insignificant menace upon the water-shed.

The disease held on through the summer months. Privy vaults abounded in the borough, also wells in close proximity thereto. People, mostly miners of foreign birth, would not heed the admonition to boil all water, and to disinfect all dejecta. Consequently, the wells were poisoned and the percentage of secondary infection was large.

The general lesson taught was startling and broad. Since one case of the disease imported could pollute the water supply of an entire town sufficiently to produce wholesale sickness and death, the officers of public and private corporations all over the country were startled at the proposition that no municipality taking drinking water from surface sources is safe.

Mr. Snow.

The conclusion reached at that time was that the individual in the village or rural district must dispose of his excrement in such a manner as not to pollute, directly or indirectly, any surface or well water.

The remedy was accepted as ideal, and impractical of accomplishment. To-day, with the aid of advanced methods of water and sewage purification, of public hygiene and municipal sanitation, a very much nearer approach to the ideal is practicable than in 1885. To this end the Pennsylvania State Department of Health was reorganized by the law of 1905, and the responsibility for progress in this branch of State medicine was placed in the hands of the Commissioner of Health.

The instances herein cited serve as a foundation for the enunciation of the principle that no unfiltered surface water, in any way likely to be polluted by sewage, is absolutely safe for drinking purposes. The risk attendant on the use of it may be slight. Where the risk is great, the life and health assurance afforded by filtration of the water is demanded as a public safeguard.

Still further, filtration is not an absolute safeguard, more especially when the source is subject to sewage pollution. The interests of the public health demand that sewage pollution of public supplies shall cease. Standards of excellence in the design, construction, and operation of water filter plants are first essentials to guard the public health, but this safeguard should not put a premium on the use of the watercourse as a sewer by some up-stream municipality. On the contrary, the original pollution of the water must be minimized.

A year ago the proper authorities of the three sovereign States—New York, New Jersey and Pennsylvania—met to formulate a policy of co-operation with respect to the sanitary conservation of inter-state streams and their tributaries. It was unanimously agreed that water filters were not alone sufficient, but that both water and sewage purification plants are demanded, in the interests of public health.

The right of eminent domain in Pennsylvania has been taken away from public water companies. It has been decided to be contrary to public policy to permit such companies to acquire thousands of acres of land for protective purposes. Further, it has been observed that purchase and control of large water-sheds does not necessarily prevent pollution.

A certain quantity of sewage is bound to find its way into the waters of most inhabited areas, but the legalizing of a certain amount of pollution of such waters, relying upon filtration to correct the effects thereof, is a dangerous expedient to practice generally. Start with the proposition that, after all is done that may

be done to keep sewage out of drinking water, the streams draining populous areas will still be unfit to use unless the water be filtered, and, using this as an argument against sewage purification works, grant licenses to towns to discharge sewage into the watercourses, and inevitably the pollutions must exceed the "certain amount." Mr. Snow.

The popular notion that it is permissible to discharge sewage into a stream which is not thereby made offensive to the sense of sight and smell is far from being dispelled. The local authorities of many towns are honestly dense with respect to this subject. It took an epidemic to teach the 1905 Board of Health of Nanticoke that the excrement of one individual could be transmitted many miles and kill those who drank the poison, although nothing might be visible in the water to indicate the danger. Yet Nanticoke is nearly opposite Plymouth. After the lapse of twenty years, the Plymouth lesson had to be repeated in the same district. It is not from the standpoint of nuisance, but from the standpoint of pathogenic pollution, that the campaign against putting sewage directly or indirectly into the waters of the State is being waged in Pennsylvania.

When the enormity of the crime of defiling water supplies appeals generally to everybody, there will be voluntary reforms. Meantime, the administration of law is a very pertinent matter. There is no public work of more importance to health than water purification and sewage disposal. Pennsylvania provides for central supervision of these utilities. No water-works or sewerage system can be legally extended or installed unless plans therefor have been submitted to and approved by the Commissioner of Health. Respecting the discharge of sewage, by a municipality, into any stream or body of water, the unanimous agreement of the Governor, Attorney-General, and Commissioner of Health is required. A penalty of $50 is provided for every day that a municipality pollutes illegally any of the waters of the State.

Dr. Charles B. Penrose, of Philadelphia, conceived the new State health laws, prepared the bills, and labored assiduously for their enactment. They are a worthy monument to a worthy man. The Honorable Samuel G. Dixon, who, as first Commissioner of Health, has had the task of organizing the new Department to put his conceptions of State medicine into operation, has inaugurated a policy which is bound to reduce water-borne diseases to a very large extent.

Every minor source of pollution on the water-sheds is to receive attention, and all menaces are to be permanently abated. Patrols by private and municipal corporations are to be required. The owner of each estate must cease to be careless about the disposal of sewage. Such disposal must be accomplished in a way unharm-

Mr. Snow. ful to others. Ordinary farm operations cannot be interfered with, but the effort is to confine pathogenic material within bounds. A health officer has been assigned to every municipality, to investigate, report, and execute orders.

Major pollutions, such as municipal sewerage systems, require more consideration. Often, the problem is one of finance. Slowly but surely the sewage of towns will cease to be discharged, untreated, into the waters of the State. First-class operation and maintenance of plants which have been constructed under State approval will be required. The necessity for State supervision is recognized, in fact, demanded. Without it, the "certain amount" of pollution of surface water supplies would be a large and dangerous amount, and the moneys, therefore, expended in the construction of sewage disposal works would be lost to a considerable extent.

Mr. Clarke. D. D. CLARKE, M. AM. SOC. C. E. (by letter).—That the purity of the water supply of any city is a matter of vital importance is so generally recognized that it seems almost superfluous even to state the fact; but, when it comes to the question as to the best method of securing and maintaining the purity of the supply, there is a diversity of opinion. The writer holds that it is better to secure such a supply as shall be of unquestioned purity, rather than attempt the purification of waters known to be contaminated in the slightest degree; but it is recognized that the financial question involved is often a controlling factor, making necessary the adoption of methods which otherwise would not be approved.

As an illustration of what it is possible to accomplish, at least in the more recently settled portions of the continent, the writer would cite the case of Portland, Oregon, a city of 150 000 inhabitants, which secures an abundant supply of pure water from an absolutely uninhabited water-shed situated on the western slope of the Cascade Mountains from 30 to 50 miles distant from the city.

The advantage of this supply, as a factor in the life and growth of the city, can hardly be over-estimated, and it is safe to say that it is with a growing appreciation of its value that the residents of Portland regard everything that pertains to it or affects it in any way; and hence they are fast coming to feel in a measure justified in felicitating themselves upon their good fortune in being permitted to dwell in a city possessing such an abundant supply of pure water.

It is now nearly twenty-five years since the question of municipal ownership began to be discussed, and surveys were made looking toward securing a water supply for the city from a mountain stream.

In 1886 a citizen's committee, numbering fifteen of the substantial business men, was empowered by the State Legislature to

purchase or construct and operate water-works for the supply of the city. In the following year the Water Committee purchased the plant of the old Portland Water Company, which had been supplying the city with water from the Willamette River by pumps installed at a point on the river bank about 6 miles above the city; these works they continued to operate until the Bull Run gravity system was completed in 1895. Mr. Clarke.

The surveys for the gravity system, made under the direction of the late Isaac W. Smith, M. Am. Soc. C. E., who also constructed the works, developed the fact that the most available and altogether satisfactory source of supply was to be found in the water-shed of Bull Run River, in the foothills of the Cascade Mountains, eastward from Portland. The point finally selected for the intake is situated on the Bull Run River, about 5 miles from its junction with the Sandy, a tributary of the Columbia, and 30 miles from Portland, the elevation of the intake being 720 ft. above low water in the Willamette River. This point is in the edge of the foothills but well above the general level of the valley, while thence eastward the ground rises rapidly to the summit of the Cascade Range.

The entire western slope of the range is covered with a dense forest, and the work of exploration is difficult at all times. Little was known regarding the water-shed of the Bull Run prior to the time of making the surveys for the city. From the maps on file in the office of the United States Surveyor-General, the approximate area of the water-shed was known to be about 140 sq. miles. The existence of a lake near the summit of the range was also known, and but little other information was available concerning the region east of the point selected for the intake to the conduit. It should be stated, however, that prior to the selection of this stream, as the source of supply for the city, the main stream and the lake were explored by engineers in the employ of the Committee, and it was thereby determined that the flow of the stream was ample to provide for the future needs of the city.

In 1891 additional explorations were made by Mr. Frank T. Dodge, Superintendent of the Water Department, and, as a result of his recommendations, the Water Committee made formal application to the Department of the Interior for the withdrawal of the entire region as a Government reserve. This withdrawal was accomplished on June 17th, 1892; under the name of the Bull Run Forest Reserve, covering an area of 222 sq. miles and entirely surrounding the water-shed of the main Bull Run River.

Regarding this reserve the following statement is made:

"The United States will not sell any land in this reserve, nor permit timber to be removed from it, nor allow sheep or cattle to

Mr. Clarke. be pastured thereon. So long as the laws governing these matters remain unchanged, the water supply of Portland cannot be injured, providing the law to prevent forest fires on the public domain is properly enforced. The forest shades the snow which every winter covers the land deeply, and retards its melting; the undergrowth protects the soil and prevents it from being washed into the streams. The result is that no settling reservoir is needed, the water is always clear as crystal and free from sediment even during the greatest freshets. It is so cool that it does not need ice to make it palatable."

At the time the Bull Run reserve was established there were within its borders but few tracts claimed by private parties, and these were mostly held as timber claims by non-resident owners. Only two or three of the claimants ever made any attempt to build houses or make any permanent improvements in the way of clearing and cultivating the land. In a very few years the Water Committee succeeded in acquiring control of all improved claims, and has since continued the work of extinguishing private title to all lands within the reserve. At the present time the entire reserve above the intake is absolutely uninhabited.

From the beginning it has been realized that the greatest danger which threatens the water supply is that due to forest fires, which are very destructive during the summer and early autumn of especially dry seasons, these fires being usually due to the carelessness of hunters, fishermen and campers within the reserve or along its borders. In order that the danger from this source may be reduced to a minimum, the reserve is patrolled by forest rangers employed by the Government, and, in addition, extra patrolmen are frequently employed and paid by the Water Department, in order that a thorough inspection and isolation of the district may be maintained. This work is in charge of the officials of the United States Forestry service. In furtherance of this protection work a special Act of Congress, passed April 28th, 1904, prohibits all unauthorized persons from entering the reserve, all trespassers being liable to a fine not exceeding $500.

The low-water flow of Bull Run River is estimated to be from 65 000 000 to 70 000 000 gal. per day. If a larger quantity should ever be required, it would be a question of storage in Bull Run Lake or elsewhere. Rainfall observations, at the head-works, at the western border of the reserve, show a yearly precipitation of about 70 in., while, at the higher or eastern portions of the reserve, it is undoubtedly much more. It will thus be seen that the average run-off from the rocky and precipitous Bull Run water-shed, of 140 sq. miles, must largely exceed any probable requirements. It is estimated that Bull Run Lake alone will afford ample storage for many years to come. By a comparatively short tunnel, the lake

could be drawn upon so as to supply an extra 100 000 000 gal. per day for the possible 60-day period when the stream is at its lowest stage. Mr. Clarke.

By special Act of the State Legislature, passed in 1906, the entire flow of Bull Run River is secured for the exclusive use of the City of Portland.

The following is an analysis of a sample of Bull Run water taken from the stream after a heavy rain:

Total Fixed Ingredients.

"2.7 parts in 100 000 parts by weight, or 1.89 grains per imperial gallon (of 10 pounds avoirdupois) consisting of:

	Parts in 100 000.	Grains per gallon.
"Silica	0.56	0.392
Oxides of iron and aluminum.	0.08	0.056
Calcium carbonate	0.39	0.273
Magnesium carbonate	0.27	0.189
Chlorides, sulphates and carbonates of alkali	0.40	0.280
Organic matter	1.00	0.700
Total	2.70	1.890

"The carbonates of lime and magnesia are in the form of soluble bicarbonates. The organic matter is mostly in the form of suspended vegetable substances and partly insoluble products of their decomposition. Microscopic examination of the sediment, and chemical tests of the water show the absence of deleterious organic matter. Only traces of ammonia were found, while nitrates and nitrites could not be detected, showing absence of nitrogenous organic matter. To sum up, the water ranks among the best on record for domestic use."

GEORGE A. SOPER, M. AM. SOC. C. E. (by letter).—It is by no means clear what sort of discussion was desired by the framers of this question. Its wording suggests that, in order to have clean water, we must either build filtration plants or purchase the land through which the water flows. This, of course, is not true. Mr. Soper.

In the average case, people using impounded supplies need not feel that they are driven upon either horn of this dilemma—certainly not to the purchase of whole water-sheds. This is at once the most expensive, least scientific, and most impracticable way to proceed in order to protect a water supply against pollution. For example, it would cost New York City about $58 000 000 to buy the drainage area of its new water supply, provided it could be had at the average rate of $100 per acre; and mere ownership is by no means an insurance against pollution.

The question suggests that pollution may perhaps be tolerated if only filtration is used. This is utterly wrong. It is wrong in

Mr. Soper. principle and not justified in practice. Filtration is a cure, and an excellent cure, as cures go, but it is not an infallible safeguard. Deliberately to allow filth to pollute a water supply with the idea that the water will eventually be cleaned by filtration shows a beautiful trust in filters and a greater confidence in the men who run them; but it is not good sanitary engineering. It has been well said that what we need in water is not repentance but innocence, and when it is possible to keep water pure this should be done by all means.

The extent to which water supplies can be protected against pollution depends upon a number of circumstances, but chiefly upon the amount of attention which engineers and others interested in pure water are willing to give to the subject. If waterworks people would give an amount of attention to regulating the drainage, inspecting the streams, and superintending the disposal of harmful matters on the water-shed commensurate with the time and money which they give to handling the water after they get it, nearly all danger from impounded supplies could be avoided.

Protection means a number of things which the public looks to engineers to understand, but which engineers frequently consider can be understood only by medical men; but physicians, as a rule, are not competent to handle sanitary matters. It is time that engineers accepted the responsibility, and undertook to deal with these subjects themselves.

The sanitary protection of a drainage area means, first, knowledge of the conditions where permanent and accidental defilement may take place; second, the careful design of measures to protect the water against these danger spots; third, adequate provision of law to carry out the measures; fourth, a settled intention to protect the water.

If the purpose exists, difficulties respecting the other requirements will disappear. Experience shows that filth can be kept out of water, if only there is a determination to keep it out.

Mr. Baldwin. WILLIAM J. BALDWIN, M. AM. SOC. C. E.—Some 25 or 30 years ago small filters were recognized as propagating beds. The speaker would like to ask whether there is anything in this, and what engineers know about it?

Mr. Williams. GARDNER S. WILLIAMS, M. AM. SOC. C. E.—It may be proper to say a word in regard to the kinds of bacteria that are to be dealt with, particularly in view of the statement of one of the speakers, that owing to the absence of light the number of bacteria may be very decidedly reduced in passing through a water-works system. It is to be borne in mind that those bacteria which are most dangerous to the human organism thrive in the darkness, to most of them

light is germicidal, so that the above-cited condition would fail almost entirely to reduce the forms of bacteria which are in any way dangerous to health. Mr. Williams.

The subject of railroad pollution is certainly a serious one, as it can readily be seen that closing the closets while traversing water-sheds might, in time, entail closing them altogether, and it seems that the more rational method of dealing with the question is either to provide receivers upon the cars for the dejecta, or to install such systems of drainage along the railroads as will divert the contamination from the water-shed.

It may be quite safely stated that while there is no question that a great deal can be done to improve a water supply by eliminating the sources of evident pollution from the water-shed, yet no such system will ever be perfect. It will never be possible to remove all such sources, and while the elimination of as many of them as possible is to be advocated, a community is not thereby warranted in using a surface water subject to possible pollution, without filtration. On the other hand, the mere filtration of the water or the mere existence of a filter operated by anybody is not a reliable safeguard. Whether the filter be of the slow-sand, or of the so-called rapid or mechanical type, it needs very close attention, and unless that careful watchfulness accompanies the operation, one cannot depend upon it any more than upon the ownership of a water-shed. A filter is a very delicate machine, and it must be carefully and skilfully handled.

It can be quite definitely asserted that, for a properly operated filter, there is no such thing as its becoming a propagation bed. The mechanical filter which is not washed at proper intervals will, of course, permit of the passage of bacteria by becoming filled, since the sand bed in such a filter operates simply as a strainer, and when the strainer is filled the material washes out at the bottom. In a slow-sand filter, however, there is no record known to the speaker of the condition suggested, in the 70 years' experience with that type; and with a mechanical filter, properly attended, nothing of the sort has ever occurred.

J. T. Noble Anderson, M. Am. Soc. C. E.—The speaker wishes to call attention to a point in Mr. Fuller's discussion, which must be due to some misapprehension on his part. He says that the water-sheds of several cities, among which he mentions Manchester, England, have been acquired at a cheap price. The speaker was employed many years ago on the Manchester water scheme, and also on that of Belfast, Ireland, and knows that the prices paid at both places were not what would be called cheap; for example, much of the Manchester water-shed was acquired from the Fleming family, who asked $2 500 per acre for the land. The Manchester Corporation offered $100 per acre —the great difference is manifest—and it was purchased for from $400 to $500 per acre. Mr. Anderson.

Mr. Anderson. In the case of the Belfast water-works the average price originally paid for the land was about 10 shillings per acre per annum, which was the rental price paid by the farmer. This, with some other items, made the net return of the land required by the Belfast Corporation about $5 per acre per annum. With the compensation paid to the tenant and that paid to the landlord, the land cost the Belfast Water Board on an average of from $200 to $250 per acre.

There is another point, which has come up in Australia, which the speaker thinks has been generally overlooked by writers on hydraulic subjects, and that is, the use of the acquired water-sheds for re-foresting, especially in sub-tropical countries. The cutting down of forests interferes greatly with the rainfall, therefore it is not only important, in many countries, to protect the water-shed from impurity, but also to protect it from natural conditions, so that the rainfall of the range in which the water-shed is situated will not be impaired. By the judicious planting of these water-sheds that result is obtained, and the rainfall may be actually increased and a revenue earned. In the State of Victoria, the City of Melbourne has a water-shed roughly estimated at 100 sq. miles of forest, that will suffice to supply a population of 750 000, which it is expected will be reached within the next 8 or 10 years, and the Government has further reserved 250 000 acres all of which will be conserved ultimately and used by the Melbourne Water Commission. The point is an exceedingly important one on the subject in hand.

Mr. W. B. Fuller. William B. Fuller, M. Am. Soc. C. E.—The speaker believes that all upland water supplies should be filtered, even though filtration is put in merely as a safeguard. Where it is suspected or known that pollution exists on an upland water-shed, the proper course to pursue is, first, to put in a filter, and then, if further funds are available, to buy up and remove the polluted portions of the water-shed.

While it is true, as has often been said, that "innocence is better than repentance," it must be recognized at the outset that no upland water supply, about which there could be any question regarding the necessity of buying up any portion of its area, can be pure; and unless the entire area is inaccessible to Man and beast, we have the ever present possibility of outside contamination of even the purest supplies.

Some of these possible sources of pollution have been pointed out, notably the contamination possible from railway trains traversing the water-shed and the need of keeping the closets closed during such times.

Passing over the difficulties and inconveniences attending the carrying out of such orders and the dangers present, supposing negligent attention on the part of the railroad officials, there are other sources of pollution of an inhabited region still more difficult to regulate, as, for instance, during the repair, construction, or reconstruc-

tion of railroads and roads where large numbers of laborers pollute the adjacent fields, the construction of country estates, the pollution by campers, picnickers and other nomadic hordes, and many other possibilities. Mr. W. B. Fuller.

It is true that large impounding reservoirs assist greatly in reducing the gross and accidental pollutions mentioned, but the standard of purity now demanded in many communities will not stop at one possible means of correction, but demands the best correction that engineering science can offer. To this end the best and safest guard against both accidental and permanent pollution is a properly designed and scientifically operated filter, the filtered water passing directly into covered conduits and not again exposed to the air, or possible contamination, until it reaches the tap of the consumer.

In advocating filtration the speaker does not wish to be understood as considering it a sure specific for all water evils, and he certainly advocates, first, the securing of the purest water supply from the least polluted upland possible; and, second, the keeping of this upland supply as pure as possible before delivery to the filters. To this end, he is a firm believer in the necessity of draining all stagnant and marshy places; the removal, by purchase or otherwise, of undesirable persons and manufacturers from the water-shed; the organization and efficient maintenance of sanitary patrol; and the taking of all possible precautions against pollution. The less the filter has to do, the safer the water, but the filter should stand at the outpost and guard against enemies that slip in unawares, as well as against those that are known.

J. WALDO SMITH, M. AM. SOC. C. E.—A large impounding reservoir at the lower limit of a water-shed is, from a sanitary point of view, one of the greatest safeguards which can be provided for a water supply. The sanitary quality of a water is undoubtedly increased by long periods of storage, though the storage may have but little effect on either its taste or appearance. In fact, the physical quality of a water may be made worse by storage, and for this reason, as well as to satisfy the proper public demand for a water, clear, and pleasant to look at, filtration is necessary. Mr. J. W. Smith.

The question as to whether or not conditions in a storage reservoir can be improved by stripping its bottom and sides of most of the original organic matter is an important one. Recent studies indicate that the effect of stripping is to guarantee a somewhat better quality of water for a few years, but that, in the end, the conditions will be the same as though but little work in this direction had been done. An unstripped reservoir will improve; a stripped reservoir will grow worse until finally, if all other conditions are the same, the quality of water delivered by two such reservoirs will be identical. Even at its very best, stripping is not an absolute guaranty; vegetable growths may occur in any reservoir and can only be guarded against by thorough aeration followed by filtration.

Señores Solís and Guzmán.

Señores PABLO SOLÍS and OCTAVIO GUZMÁN* (by letter).—The writers congratulate themselves, as representatives of the State of Puebla, upon the opportunity that now presents itself of knowing, through this meeting of eminent professional men, the present state of the art in the five technical subjects which make up the programme. They regret that this programme lies without the field of professional practice of their State, which is purely textile, and in whose capital, on account of its excellent climatic and topographic conditions, it has not been necessary, up to the present time, to introduce modern sanitary devices. On account of the extraordinary firmness of its soil, it has not been obliged to contend with exceptional foundation problems; and, as the center of an extensive network of railroads which has completely done away with care of the ancient roads, it has not preoccupied itself concerning the future fate of the public highways. The recent growth of automobiling, however, now compels attention to, and interest in, them. On the other hand, our capital being on the eve of establishing a system of drainage and all manner of sanitary improvements, the subject of potable water supplies also greatly interests the State, and, through the writers, it will secure, from the deliberations of this meeting, knowledge of the latest advances in all the important matters proposed for discussion, and the writers will endeavor to co-operate by discussing the two first questions of the programme presented, giving their opinions on these subjects on an abstract basis.

With regard to the first subject under discussion, the writers believe that, with the present state of knowledge and experience in the art of sanitation, only the second alternative stated in the question is practical, that is, to permit a certain amount of contamination, and to rely on a subsequent purification.

In truth, the art of sanitation, in spite of the great advance that has been made in the acquisition of knowledge and of practice, has, up to the present time, only undertaken the sanitation of isolated and independent communities. In sanitary matters facts must be considered in their multiple condition, not isolating some with respect to others, because they are never independent. From this point of view it is of little use to attend to the sanitation of one city, if the valley or region in which it lies is not also taken care of, and in the sanitation of any region the disinterested co-operation of the various communities in it is indispensable.

Unfortunately, up to the present time, each community watches its own interests only, not concerning itself with the interests of the others with which, on account of local conditions, it is inseparably linked, and on whose healthfulness depends in great part its own. If

* Delegates, to the Convention of the American Society of Civil Engineers from the State of Puebla, Mexico. (Discussion translated from the Spanish.)

Señores Solís and Guzmán.

a community does not dispose properly of its refuse, getting rid of it without special treatment of any kind, its final disposition being near neighboring communities, the latter will become contaminated, and disease will be developed in them which may infect the culpable community and even the whole region. Therefore, sanitation, to be complete and effective, must be more than local, it must cover the entire region, and must embrace all towns that are linked by natural conditions; and as long as this desideratum of the sanitary art is not reached, the potable waters in such regions, whatever their origin may be, if they are below the level of the sewage of other towns, will be somewhat contaminated. Even granting great vigilance and control by the Governmental powers, the result will be the same, because unless such regulation is mechanical and automatic, it will be inefficacious. It is fruitless, therefore, to purchase and watch the water-sheds that feed the reservoirs, and, in the opinion of the writers, it is best to allow contamination to take place to a certain degree, trusting to filtration for purification. The question may well be asked: Is regional sanitation which will prevent the contamination of water possible in practice? Judged purely in the abstract, and by the theories of sanitary art only, an affirmative answer forces itself upon us, and, furthermore, that this is the only proper form. It is incomprehensible that any system which restricts its attention to isolated points can be called sanitary. The phenomena of life, under whatever form we may consider them, be it individually or collectively, are always the same; and as, in the individual, health cannot exist unless it exists in the whole organism, in the same way, collectively, it will not exist if it does not encompass the whole region. Now, what are the fundamental conditions necessary to regional sanitation, in the strictest conception of the term? The very same conditions as are necessary in individual life; and, summed up, they may be condensed as follows: Continuous renovation; continuous renovation by circulation or motion; and continuous renovation by chemical or substantial transformation. In the life of the individual these conditions in their admirable harmony are regulated by what is known as "the vital principle," and in collective life they are, in turn, regulated by the laws of political economy, and the rights of the individual under the direction of the Government. It is within the field of political economy to arrange financially for the problem of continuous circulation by chemical or substantial transformation, and it is the arduous task of the Government to harmonize sanitary interests in such a way that the best interests of the whole region will be preserved.

In the same degree as these difficulties are overcome, will the art of sanitation advance from being purely local, as it now is, to being regional, as it will be to-morrow, and in that same degree potable waters may be considered as safe from contamination.

Señores Solís and Guzmán.

In what degree has the scientific disposal of garbage been successfully and economically applied to the sanitation of towns? Much effort has been made in this direction, and results have been obtained, propitious in some cases, adverse in others, but thus far the advantageous results have not been confirmed by properly made estimates, and the general opinion among sanitary engineers is that we have not arrived at satisfactory results, especially with respect to sanitation by gravity, in which process the matter being highly dissolved, its extraction is very costly.

Now while the art of sanitation remains in the precarious condition of not being able to get rid of detrimental matter except by the process of solution and removal, the contamination of potable waters is inevitable, and it is necessary to resort to filtration, which is the only solution of the situation in the present state of the art. But when the economical transformation of domestic and other wastes becomes possible, to the extent that it may be applied generally, and when the mental condition, customs, and altruism of the community have advanced to the perfection required by the present state of sanitary science, it will be possible to put its teachings into practice. The decisions of Boards of Health may then be carried out with efficiency, and then regional sanitation may be accomplished, and the non-contamination of potable waters will also be an established fact. In the meantime we must be resigned to allow potable waters to become contaminated, and we must purify them by filtration.

Mr. Russell.

S. BENT RUSSELL, M. AM. SOC. C. E.—It is evident from this discussion that the question as to which procedure is the better policy cannot be met with an answer so simple and direct that it will not need much explanation and limitation as to conditions.

Mr. Fuller has set forth in an admirable way the facts which enter into the problem. He has weighed them in an impartial spirit, and his conclusions, for the most part, seem to be logically drawn. To the speaker, however, it would seem that part of his first conclusion has not been fully demonstrated in his argument. It may yet be shown that, with equal intelligence and watchfulness, the purchase and control plan is as efficient as filtration, with regard to the sanitary character of the water. The cases referred to by Mr. Snow will bear this out.

The other part of this conclusion—that in regard to the physical condition of the water—is undoubtedly sound, and is a very strong point in favor of filtration. This advantage will usually outweigh the detrimental factor of sentiment against impure water artificially made pure. The truth of this has gained more general credence with the growth of the science of water purification.

To make the involved point more clear, compare two towns, one with a water-shed owned and controlled, the other where a certain

amount of pollution is allowed, but filtration is in use. Which has the better water supply? This is still a debatable point among engineers; some favor the former, others as strongly the latter. Either plan will give reasonable safety from water-borne diseases, provided intelligent care is exercised in the management. It might be said, as in the matter of forms of government: Mr. Russell.

"* * * let fools contest;
Whate'er is best administer'd is best."

With good administration, the former town will have the better supply, from the point of view of one who thinks a naturally pure water is better than a purified water; while the latter town will be better protected usually against objectionable tastes and odors and visible impurities, as shown by Mr. Fuller. It may be remarked that a majority of water consumers do not inquire into the purity of the supply, if the water is clear and sweet. On one side, therefore, it is found to be a matter of sentiment or prejudice in favor of water naturally free from sewage pollution; on the other, that of prejudice against water which is at times objectionable to the senses. The speaker believes that of late years progress has been in favor of filtration.

If it is admitted that there is no great balance of advantage in favor of either plan as regards quality, the question of cost may then be taken up. At the final test a community usually wants a satisfactory water at the lowest cost. Any further expenditure must be more than justified, in money value, by the superior quality. It must be recognized, as Mr. Clarke says, that the financial question is often a controlling factor. Mr. Fuller shows that, in the matter of cost, filtration has the preference. In any given case, however, the cost will depend upon local conditions, and, therefore, the procedure must be governed by these conditions. Among the local conditions, existing laws and customs, as affecting sanitary control, must be considered.

Mr. Fuller states that there is need for better legislation regarding sanitary matters, and Mr. Snow shows most clearly what proper legislation can do for the protection of water supplies. This brings us to the broad question of proper legislation for controlling the disposal of sewage and wastes, and the pollution of lakes and streams. The speaker understands that, in the matter of such legislation, many of the States, and the United States, as a whole, are sadly behind the best practice of European countries.

These, of course, as far as pertains to the United States, are all questions of the future. Engineers are now working toward the adoption by the Government of wise laws for the protection of future generations against the dangers that will come with increased population, and they should continue in this work with still greater zeal and

Mr. Russell. co-operation. Such laws are required by true public economy in the matters of sanitation and water supply. Proper legislation will reduce to a minimum the expense of protecting the water-sheds.

From these considerations, with a view to future public economy and health, the following may be formulated as an answer to the question under discussion: The best policy is to limit the pollution of the water-shed by suitable legislation leading to proper drainage and disposal of wastes, followed, of course, by vigilant enforcement of the laws, so that they will effect their purpose. This should be supplemented by filtration of such character and efficacy as to produce a satisfactory public water supply, as defined by Mr. Fuller.

In the speaker's judgment, the cases requiring purchase of land instead of filtration would be so few that they could be classed as exceptional.

Whether or not this answer is accepted, all may fairly hope for, and work together for, legislation to protect our streams and lakes from pollution, and for the better observance of sanitary laws by the public.

Mr. Sherrerd. M. R. SHERRERD, M. AM. SOC. C. E.—It seems as if there might be a middle ground between the views expressed in the discussions by Mr. G. W. Fuller and Mr. George A. Soper, one of which points to filtration as the only means of safety, while the other seems to indicate that filtration is not necessary. It seems to the speaker that proper sanitary protection turns on the point of whether water-sheds can be selected which will be safe without filtration; perhaps it will be essential that, in every case, filtration must be resorted to eventually. It would seem possible, where the population is small and perhaps occupies only a portion of the water-shed, that the installation of a sewer system to take off the waste from that population, which represents the more expensive portion of the water-shed (from the standpoint of purchase), could be resorted to, and that the outskirts, which are generally the cheaper part, should be purchased, so that a combination of sanitary measures would be effective for a great many years. This would certainly be a cheaper method in many cities in the United States than the immediate installation of filtration, and would be also a very satisfactory protection for many years.

One of the discussions referred to the effect of sedimentation, and it might be of interest to give some figures as to the reduction of bacteria noticed in the water supply designed for a population of some 300 000, where the capacity is about 50 000 000 gal. daily. On this plant there was recently constructed a storage reservoir near the city holding about a month's supply. Formerly, the water was brought down through some 25 miles of pipe line, and, at the intake, the bacteria would run as high as 5 000 per cu. cm., and in the tap water of the city it would be reduced, principally due to the absence of light

in the pipe lines and sedimentation in the city reservoirs, to an average of 200 bacteria per cu. cm. After the use of the storage reservoirs, where the water is held for about a month, the bacteria being about the same at the point of intake on the water-shed and the conditions being similar, the number of bacteria in the tap water has been reduced to an average of from 20 to 30 per cu. cm., running along at those figures for months at a time. Mr. Sherrerd.

There is one other feature in the protection of water supplies which seems to be essential and to which more attention will no doubt be paid in the future; that is, the danger of pollution from railroads traversing the water-sheds. It would seem conclusive, and public health authorities have recommended, that stringent laws will be necessary to secure proper sanitation in this respect unless the railroads see that it is to their interest to provide some sort of sanitary measures. In New Jersey it was proposed that they should go even so far as to enact legislation which would put this matter in the hands of the State Board of Health. This was done at the suggestion of some of the local authorities who were interested in the matter, and at least one railroad has agreed to close the closets on the trains while traversing the water-shed of the Newark water plant. This is done also on the New York, New Haven and Hartford Railroad on the New Haven water-shed, and is being done on the water-shed of the Scranton, Pa., water supply because of an order from the State Board of Health. It was also proposed to do this at Seattle, Wash., on the portion of the road which was to be built through the territory from which that city derives its water supply.

New Jersey has enacted legislation which will allow the cities to provide sewerage facilities for the drainage of the population on water-sheds where the State Board of Health creates a district, and the State Sewer Commission approves the plans. The cities are to pay for the installation of these systems.

To insure that the sewage be treated properly before it is discharged into the stream, it is proposed, on the plan to which reference was made, that a sewer system shall be installed with septic tanks, and that broken-stone sprinkler filters and fine sand filtration shall be resorted to before the outfall discharges into the main stream below the intake. This act provides that, while the city shall pay the entire cost of installation, it may have the authority to enter on private property and cause connections to be made from outhouses or closets to the sewer, and provide proper sanitary arrangements; and it is also proposed that a water supply shall be installed from one of the reservoirs in order that the sewer may be flushed properly.

In the speaker's judgment, the purity of the water supply for this particular city will be insured, at least for a great many years. There is one question, however, which is troublesome, namely the fact that

Mr. Sherrerd. by the installation of such a system, particularly in the neighborhood of a suburban section of the metropolitan district around New York, the property, to a certain extent, is made more desirable for residence purposes, particularly for villa sites where people from the metropolis may have all those advantages, so that this has, in a measure, a tendency to increase the population of the district. On the other hand, to attempt to secure by purchase the tracts of land that are necessary to protect a water supply is usually much more expensive than the cost of installing and maintaining adequate sewerage facilities.

The conclusion of the speaker is that conditions must always govern, and that it is essential that the purest water should be obtained primarily; its treatment will depend on circumstances.

Mr. Blackford. F. W. BLACKFORD, M. AM. SOC. C. E.—This question is largely one of economics, and should be considered like any other problem that comes before an engineer. Local conditions make a great difference in such propositions. In populous districts, like those adjoining New York, and in Pennsylvania or New Jersey, and other places similarly situated, it might be impracticable to acquire a water-shed entirely, but in many places it would be practical. Of course, this depends largely on the ability of the community to tax itself sufficiently to obtain a large water-shed or one of sufficient size to furnish a supply, but it is a question of locality and a question of making figures, like other propositions, such as building railroads and adjusting the grades and curves to the amount of money available, and to the traffic which is expected. In many places, even in populous districts where the water supply is drawn from a distant point, a large tract of land could be acquired and controlled absolutely. It does not follow that such land should remain useless; it could be planted in grass or in forest—more properly in forest, which, in the course of a generation, would become profitable; and, while it might also be found necessary to filter the water to some extent, the circumstances might be such that it would not have to be filtered at all. It is an economic proposition, and one should consider the amount which the community under the law could tax itself, the amount of income to be derived, the operating expenses, etc., as against the fixed charges.

Mr. Johnson. GEORGE A. JOHNSON, ASSOC. M. AM. SOC. C. E. (by letter).—It is no simple matter to make a sizable water-shed hygienically clean and keep it in a thoroughly safe condition, either by purchase, depopulation, or sanitary patrol. The cleanest and most carefully policed water-shed is likely to receive "accidental" pollution, at one time or another, through the discharge upon it, or into the streams draining it, of typhoid-infected excreta by a chance traveler suffering from a mild case of this disease, that is, "walking typhoid." This thought as to danger in unsought places may be carried even further by directing attention to the fact that typhoid bacilli may persist in the

urine of convalescent typhoid patients for weeks and even months. Some pathologists claim to have detected this organism in the urine several years after convalescence. Mr. Johnson.

It is a fact that, in general terms, the degree of danger existing in a water supply may be measured by the cleanliness of the water-shed; in a sense, it stands in inverse proportion to it, other things being equal. There can be no doubt that an impounded supply which is filtered will always be safer than a filtered supply which is not impounded, owing to the natural purification effected in impounding reservoirs.

River waters which are derived from populated water-sheds, no matter how efficient are the methods of disposal of the wastes of the resident population, cannot be looked upon as safe in any sense, if such waters are pumped direct from the river and supplied to the consumers in an unfiltered state. Such waters may be made pure, according to existing standards, by filtration, but the degree of purity of the filtered water depends much upon the nature of the filtration process, the suitability of the plant, as built, to meet existing conditions, and, finally, but by no means least, upon the manner in which the filters are operated.

It is well known that polluted surface waters which are impounded in large reservoirs undergo a material purification due to natural agencies, the extent of such purification being greatest where the period of storage is longest. Published data show that at Washington, D. C., where the period of stay of the water in the series of reservoirs ranges from two to four days, the bacterial removal amounts to about 93 per cent. A similar removal was effected in the reservoir at Lawrence, Mass., when the average period of storage of the raw water was about two weeks. Equally good, and doubtless better, results may be generally expected in the case of large impounding reservoirs, such as the Wachusett, Croton, and Boonton, where the average storage periods have been roughly computed as about 532, 235 and 244 days, respectively. Bacterial deposition in fairly clear water is slower and generally less in amount than in very muddy water.

Such information as is available indicates clearly that typhoid germs do not multiply in water. In the course of a week or two a large majority apparently dies out, due to such causes as osmosis, sedimentation, unfavorable environment, lack of a suitable food supply, antagonism on the part of other bacteria, and to the fact that all bacteria serve as food for some of the higher organisms. It appears to be certain that a small percentage of the typhoid germs, constituting the more hardy forms, will live in water and mud for an almost indefinite period. The virility or pathogenicity of these resistant forms is something about which but little is definitely known. Whether or not their ability to produce typhoid fever is equal to the average

Mr. Johnson. pathogenicity of all the typhoid germs at the time of their discharge into water is not certain. It probably approaches such a standard as indicated by the quite lengthy list of epidemics following spring thaws, and the washing into streams of long-standing infected excreta.

Mr. G. W. Fuller. GEORGE W. FULLER, M. AM. SOC. C. E. (by letter).—The discussions on this topic appear to cover the question quite fully, as it is now understood. Probably the most gratifying feature is the almost unanimous viewpoint that a better quality of water is desired than formerly, and that, to this end, there is no doubt that better laws are required. The State of Pennsylvania has recently taken a long step forward, as described by Mr. Snow. It is also clear that in future sanitary developments, as pointed out by Dr. Soper, the engineer is likely to assume more responsibility, and to direct various details, in keeping water-sheds clean, to a greater extent than has been generally the case hitherto.

Local problems can obviously be best solved only after weighing the local evidence. In general, preference now appears to be given to that treatment which will afford satisfactory results at the least cost.

On this basis, it seems clear to the writer that there is bound to be rapid progress made in the introduction of filters for ordinary impounded water supplies, even where the storage period is comparatively great, and also in the systematic elimination of constant sources of pollution as well as in safeguarding against so-called accidental pollution.

The main difference in opinion seems to relate to a choice of the two treatments under consideration for places where the cost may be about the same. Sentimentally, the preference in the minds of many lies with the unpolluted source, and there is much to substantiate that viewpoint, especially for small water supplies. For large water supplies, the writer believes that the pollution should be eliminated as far as practicable, without the purchase of much land, and filtration should be generally adopted, both for the sake of the hygienic quality of the water and also for its general cleanliness, as mentioned by Messrs. Smith and W. B. Fuller.

It seems futile to say more about the relative merits of these two problems until further detailed evidence is available. In preparing his earlier statements, the writer gave more weight than perhaps is indicated to the belief that the filtration of so-called pure waters will produce improvements that perhaps some of the discussors do not realize. The Little Falls filter plant, treating a fairly safe upland water, has certainly brought about a supply, not only of attractive appearance, but also a surprising reduction in the prevalence of typhoid fever in Paterson and Passaic, N. J. This is in line with numerous experiences abroad. It is also the view of Health Commissioner Darlington, of New York City, with regard to the filtration of

Mr. G. W. Fuller.

the Croton supply which, for years, was considered of a good quality, according to the standards of earlier years. After the citizens of New York have an opportunity to enjoy the benefits of the recently recommended filter plant for the Croton supply, the writer is positive that the benefits will be considered as worth far more than their cost.

As to the inquiries made in the course of the discussion of this topic, it is to be stated that the propagation of bacteria, as mentioned by Mr. Baldwin, is not an unusual occurrence in small house filters which are infrequently and improperly cleaned. As stated by Mr. Williams, bacterial growths are quite unusual in large filter plants which are well built and well operated. They have occurred at times, however, both in test filters and in large filters in actual practice. During the summer of 1892 bacterial growths developed in the experimental sand filters at the Lawrence Experiment Station, as described by the writer in the 1892 Report of the Massachusetts State Board of Health. The reports to the Local Government Board of England, by the late Sir Edward Frankland, show that bacterial growths developed in some of the London sand filters during the winter of 1894-95. Bacterial growths were found in the experimental mechanical filters at New Orleans, as stated in the report of R. S. Weston, Assoc. M. Am. Soc. C. E., upon his investigations made in 1901.

Mr. Anderson refers to the land costs of the water-sheds of British cities: These costs do not seem to the writer to be very high, but they are much in excess of recent figures, stated to him as paid for other and presumably more lately acquired water-sheds.

Mr. Sherrerd's references to the experience of Newark are all of interest and importance, and emphasize the fact that local conditions have to be carefully weighed. Without doubt, large storage reservoirs away from the water-shed and the purification of all sewage on the sparsely populated water-shed are helpful, as is also true of the natural aeration of the water drawn from near the bottom of the Newark impounding reservoirs. In the absence of evidence of such water being prejudicial to health, and of being objectionable to the senses, it is not likely that large existing impounded supplies of that type will be generally filtered until more evidence is obtained to show the benefits of filtration as applied to such cases, as regards the improvements effected in the physical character of the water and as to its sanitary quality.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 1060.

GAS ENGINES.

An Informal Discussion at the Annual Convention, July 10th, 1907.

SUBJECT FOR DISCUSSION:

"(*a*) What is the best apparatus and most economical system for cleaning producer or furnace gas, to be used in gas engines?

"(*b*) To what extent is ordinary producer gas, made from bituminous coal, used in gas engines, and what practical results have been obtained by any methods for removing tar or soot?"

BY MESSRS. JAMES CHRISTIE, ROBERT HEYWOOD FERNALD, TIMOLEON GEELEN, AND J. M. BUCHANAN.

Mr. Christie.

JAMES CHRISTIE, M. AM. SOC. C. E.—When furnace or producer gases are to be used in engines, it is necessary that the dust, or tarry matter, in the gas, be removed entirely or nearly so. It is also necessary that sulphurous gas, if present to an objectionable excess, be reduced, or that the engine and its accessories be designed and constructed so that corrosion or electrolytic action will not ensue.

The necessity of clean gas in the engine is so important that scrubbers or washers, of the form and type heretofore used in connection with regenerators, etc., have not proved generally satisfactory, and, as a consequence, some trouble has developed in engines using gas insufficiently cleaned.

The gas must also be sufficiently cooled, and it is desirable that it should contain as little moisture as possible, as water vapor reduces its efficiency; and, as the various forms of washing apparatus

use a spray of water to remove the dust, the gas at the engine will be saturated, to a greater or less extent, depending on its temperature, unless special filters or vapor-absorbing devices are used. Mr. Christie.

Washing apparatus can be divided into four general types as follows: The baffle system; the coke scrubber; and two mechanical methods, one in which slowly revolving wetted surfaces are presented to the flowing volume of gas; the other a high-speed machine depending on centrifugal action. In blast-furnace practice, the usual dust catchers remain, as a preliminary, to reduce the work of the washer.

The first named types are so well known (or descriptions thereof are so readily accessible) that it is unnecessary to give their details here. The several mechanical washers are controlled by patent rights, therefore reference for description should be made to the manufacturers. It appears, however, that apparatus of the centrifugal type, especially when required to be of large capacity, as in blast-furnace practice, have proved highly efficient, and economical in space occupied and in water used. In recent American practice a removal of 97% of the solid impurities is reported.

The size and cost of the various apparatus depend on the amount of solids to be removed from the gas, and on the degree of purity required. Some manufacturers claim to remove the dust, so that not more than 0.01 gr. per cu. ft. of gas will remain. In a general sense, and for washers of considerable capacity, it may be assumed that the water consumption will vary from 10 to 25 gal. per 1 000 cu. ft. of gas washed; and the power required for mechanical treatment, from 0.10 to 0.20 h. p. per 1 000 cu. ft. of gas per hour, according to the amount of impurity present.

In producer practice, for power purposes, the fuel used has heretofore been confined to anthracite or other non-coking coal, and the coke scrubber is generally used for gas cleaning with the water spray for washing, and a supplementary coke chamber without water for drying the gas.

The abundance and cheapness of natural gas, throughout such a large area of the Middle States, has hitherto prevented any development of bituminous coal producers for power gas. The constant poking required to break up the coke, and the presence of either tar or soot or both, renders the production of gas with bituminous coking coal a more difficult problem than when non-coking fuel is used.

Recently, however, some promising work has been done with coking coal, and the experiments made at St. Louis under the direction of the United States Government have been instructive.

When bituminous coal is gasified in the producer under low pressure and slow combustion, say with a coal consumption of less

Mr. Christie. than 10 lb. per sq. ft. of fire bed, the quantity of tar suspended in the gas is comparatively large, the quantity of soot is small, and the temperature of the issuing gas is low. On the contrary, with more active combustion, when the consumption of coal is from 20 to 30 lb. per sq. ft. of fire bed, as in some mechanically-operated producers, the quantity of tar is small, or entirely disappears, the discharge of soot or lampblack is greater, and the temperature of the gas is much higher.

The quality of the gas appears to be about the same in each case. In addition to the soot carried over in the latter case, a considerable quantity of coke dust frequently appears; this comes from the fine particles of coal which fall through the atmosphere of high-temperature gas and are light enough to be wafted forward in the issuing stream of gas.

In modern producer practice, considerable quantities of steam are used advantageously—more than 1 lb. per lb. of coal consumed is common practice—with a result that the gas contains from 25 to 30% of its total volume of CO, or 10 to 15% of hydrogen. The steam also prevents the formation of hard clinker, and facilitates the discharge of the ash, the limit in its application being the amount that can be used without dampening combustion, or that can be decomposed in its passage through the incandescent fuel.

For the economical use of this gas in engines, it becomes necessary to provide apparatus which will cool the gas, utilize the extracted heat, remove or otherwise utilize the tar, remove and utilize the soot, and dry the gas after it has passed through a treatment with water.

The utilization of the sensible heat of the gas has been accomplished successfully by applying the heat to generate steam for the producer, or for preheating the air supply, or for both. Various methods of cleaning are proposed, but are still in the experimental stage. The tar problem has been handled in two ways: Separation by centrifugal action, as accomplished in the Government experimental plant at St. Louis;* and by the decomposition of the tar by passing the gas through a supplementary bed of incandescent anthracite or coke. The latter method is reported as being applied successfully in Germany to fuels of the lignite class, and has been applied to a limited extent in the United States, especially in the production of composite gas, which forms a specialty with manufacturers of power-gas apparatus.

This latter method of utilizing the tar has been accomplished by various methods, which differ in the details of application, and are claimed to be covered by patents. It appears, however, to be the most successful system for the complete disposal of tar, except

* Bulletins 48 and 261, U. S. Geological Survey.

by the use of auxiliary attachments to the mechanical cleaner, which require attention and renewal. Mr. Christie.

Obviously, the respective economies of the two methods depend somewhat on the demand for and value of tar in the vicinity. As the weight of the tar will be from 5 to 10% of the coal consumed, and sometimes much more when lignite is used in the producer, its salable value is an important factor in the problem.

When a large quantity of soot is formed, its removal has always been difficult, as it mixes with water very reluctantly in the scrubber, and is apt to create a nuisance, by the trouble involved in handling it, or by its obstruction of the gas passages. Sulphurous constituents in the gas have been dealt with by passing the gas through a mass of oxidized iron cuttings, a process which involves frequent renewal of the iron, if the content of sulphur is high. It is believed, however, that trouble from this source is not serious, if proper precautions are observed in the engine to avoid contact between materials which may promote corrosion or electrolytic action.

Altogether, it cannot be asserted that the generation of producer gas for power purposes, from bituminous coal, is yet in a satisfactory state of development. It will be understood that no reference is made here to the by-product recovery system, which is a distinct feature of producer practice.

ROBERT HEYWOOD FERNALD,* ESQ. (by letter).—In presenting the following discussion, the writer's remarks are confined to the utilization of producer gas for power purposes in the United States. Mr. Fernald.

Rapid Development of the Gas Engine.—It was during the latter part of the nineteenth century that the gas engine found its way to the market, and, although many types have been produced in the past 20 or 30 years, it is only within the past 5 or 6 years that the development of large engines has been noted. This development started in Germany, Belgium, and England some 8 or 10 years ago, but marked progress has been limited to the past 6 years.

For many years the natural fuel of these internal combustion engines was city gas, but even this was too expensive except for engines of small capacity, and it was seldom found feasible to operate engines of more than 75 h. p. on this fuel. Cheap gas was essential for the development of the gas engine, but the early attempts in this direction were somewhat discouraging, and, for a time, the probability of encroaching to any extent upon the field occupied by the steam engine was very remote.

The theoretical possibilities of the internal combustion engine

* Professor of Mechanical Engineering, Washington University, St. Louis, Mo., and Engineer in Charge of Gas Producer Division of the Technologic Branch, United States Geological Survey.

Mr. Fernald. operated upon cheap fuel promised so much that the practical difficulties were rapidly overcome, with the result that steam boilers and engines in many plants were replaced by gas engines, and, at the present time, the internal combustion engine is rapidly becoming a serious rival of the steam engine in many of its applications.

The development of the gas engine in point of size has been exceedingly rapid. It was only 7 years ago that a 600-h. p. engine, exhibited at the Paris Exposition, was regarded as a wonder, but, four-cycle, twin-tandem, double-acting engines to-day run as high as 6 000 h. p.

Development of the Gas Producer.—This rapid advance of the large gas engine has been made possible by the great strides which have been made in the production of cheap gas directly from fuel through the aid of the gas producer. An early form of producer introduced in Europe, and one that is in very general use to-day, both abroad and in the United States, is known as the suction producer, named from the fact that the engine develops its charge of gas in the producer by its own suction stroke. Although this type of producer is used very extensively, its use is confined to small installations, usually not exceeding 150 h. p. As far as known, the first suction producer operated in the United States was installed in 1903, although other types of producers were tried in this country a few years earlier. A serious limitation to producers of this type has been the fact that, owing to the manner of generating the gas, no tarry fuels could be used, thus barring bituminous coals, lignites, peats, etc. The fuels in most common use for suction producers are charcoal, coke, and anthracite coal, although attempts are being made to construct suction plants so that they can be operated on the bituminous, or tarry, coals.

In order to meet the demand for the concentration of power in large units, instead of operating a large number of engines of small power capacity, the pressure producer came into existence. This producer develops its gas under a slight pressure, due to the introduction of an air and steam blast, and the gas is stored in a holder until such time as demanded by the engine. Owing to the fact that the gas may be stored in a tank before passing to the engine, and also because the gas is produced under pressure and does not depend on the suction of the engine for its generation, it is possible to introduce the necessary machinery for the removal of tar and other impurities from the gas, thus opening the way to the use of bituminous coals and lignites. Following closely upon the pressure plants came the down-draft producers, which fix the tar as a permanent gas and, therefore, can use the bituminous coals and lignites.

These pressure and down-draft plants have been in operation for

the past few years, but, in most cases, until very recently, the fuel has been anthracite coal, although occasionally plants ventured on the use of a few well-tried bituminous coals, known to be especially free from sulphur, and low in ash and in tar production. It remained for the United States Geological Survey, in its testing plant at St. Louis, to attempt the use of any and all bituminous coals, lignites, and peats, without reference to the amount of sulphur or tarry matter found in the fuel. It is gratifying to note that every coal received has been run through the producer, and that the results have been more than satisfactory. Mr. Fernald.

Number of Installations in the United States.—Within the past 2 or 3 years, the development in the gas-producer industry has been very rapid. At the present time there are probably twenty companies in the United States manufacturing gas producers for power purposes. At least twelve of these are fully established on a commercial basis and are in position to give proper guaranties when installing plants.

About one hundred and fifty gas-producer power plants, ranging in size from 20 to 6 000 h. p., are now in operation in the United States. One company alone reports twenty odd installations, averaging more than 2 000 h. p. each, and nearly as many more, averaging about the same size, contracted for or now being erected. The horse-power represented by these one hundred and fifty installations amounts to about 70 000.

The number of installations and the persistent development have already led the National Board of Fire Underwriters to issue special rules and requirements for the "Construction, Installation and Use of Coal Gas Producers (Pressure and Suction Systems)."

Of the total number of installations in the United States, it is interesting to note that about two-thirds are suction plants which operate on anthracite coal, charcoal being used in a few cases. Bituminous coal is used in approximately one-third of the installations, but this one-third of the number of plants probably covers in the neighborhood of from 65 to 75% of the aggregate horse-power rating, or in the neighborhood of 50 000 h. p.

Tests of the United States Geological Survey.—This gas-engine, gas-producer problem, with its possibility of driving out the steam engine in many large installations, has become so extremely important that the United States Government has made special provision for producer-gas tests at its coal-testing plant originally installed in connection with the Exposition at St. Louis. These tests have resulted in furnishing very valuable data on the relative consumption of coal per horse-power per hour when used by the steam plant and by the gas plant.

The steam plant upon which the tests were made consists of two

Mr. Fernald. 210-h.p. Heine boilers which furnished steam for a 250-h.p., simple, non-condensing Corliss engine. This engine was belted to a Bullock electric generator.

The producer-gas plant is a Taylor pressure-gas producer furnished by R. D. Wood and Company, of Philadelphia. This producer, designated as No. 7, is of 250 h. p. capacity. Together with the producer, are the usual economizer, scrubber, tar extractor, purifier, and holder.

The gas engine installed is a three-cylinder, vertical, Westinghouse gas engine, with cylinders 19 in. in diameter and having a 22-in. stroke, rated at 235 b. h. p. on producer gas.

The engine was belted to a 6-pole 175-kw. Westinghouse direct-current generator. The load on the generator was controlled by, and the energy developed was dissipated through, a water rheostat especially constructed for the purpose.

After installing this plant in 1904, one hundred and sixty-two producer-gas tests were made, prior to moving the plant to the Jamestown Exposition grounds, in March, 1907. The fuels used have been bituminous coals, lignites, and peats, from twenty-six different States, as indicated in Table 1.

TABLE 1.—COALS, LIGNITES, AND PEATS TESTED BY THE UNITED STATES GEOLOGICAL SURVEY AT ITS ST. LOUIS PLANT.

	Number from each State.	
BITUMINOUS COALS TESTED:		
Alabama	3	
Arkansas	2	
Illinois	29	
Indiana	15	
Indian Territory	2	
Iowa	1	
Kansas	2	
Kentucky	5	
Missouri	1	
New Mexico	3	
Ohio	10	
Pennsylvania	13	
Tennessee	8	
Utah	1	
Virginia	5	
Washington	1	
West Virginia	14	
Wyoming	5	120
LIGNITES TESTED:		
Arkansas	1	
California	1	
Colorado	1	
Montana	3	
North Dakota	4	
Texas	4	
Washington	3	
Wyoming	1	18
Carried forward		138

	Number from each State.	
MISCELLANEOUS FUELS TESTED:		
Brought forward		138
Argentine	1	
Brazil	1	
California and Cinders	2	
Florida Peat	1	
Massachusetts Peat	1	
Rhode Island Anthracite	1	
Virginia Pea Anthracite	1	
Coke Breeze	1	
Miscellaneous Refuse	1	
Coke	1	11
DUPLICATE TESTS OF COALS:		
Illinois	2	
Indiana	4	
Kansas	1	
Ohio	1	
Pennsylvania	2	
Tennessee	1	
West Virginia	1	12
DUPLICATE TESTS OF LIGNITES:		
North Dakota	1	1
Total tests made		162

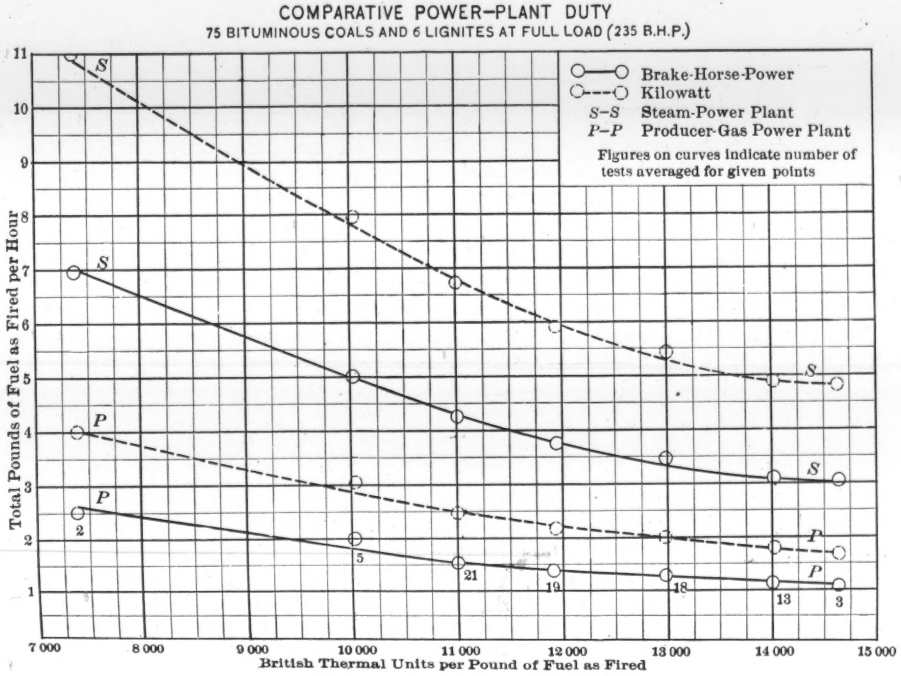

COMPARATIVE POWER-PLANT DUTY
75 BITUMINOUS COALS AND 6 LIGNITES AT FULL LOAD (235 B.H.P.)
Brake-Horse-Power
Kilowatt
S–S Steam-Power Plant
P–P Producer-Gas Power Plant
Figures on curves indicate number of tests averaged for given points
Total Pounds of Fuel as Fired per Hour
British Thermal Units per Pound of Fuel as Fired
11
10
9
8
7
6
5
4
3
2
1
7 000
8 000
9 000
10 000
11 000
12 000
13 000
14 000
15 000
S
S
S
S
P
P
P
P
2
5
21
19
18
13
3

FIG. 1.

Mr. Fernald.

No Injurious Effects from Sulphur.—Especial attention is called to the fact that, during all tests conducted at this plant, no attempt was made to remove the sulphur from the gas before it entered the engine. Considerable controversy has arisen in various parts of the country regarding the influence of sulphur upon the cylinders of a gas engine, leading, in some cases, to the introduction of this question into important lawsuits. The engine in use at the testing plant has received the full charge of sulphur contained in the gas, since the establishment of the plant, and shows absolutely no signs of injurious effects, although coals containing as much as 8.1% of sulphur have been used.

Relative Results of Steam and Producer-Gas Tests.—In considering the relation between the economic results of the two types of plants under discussion, namely, steam and producer gas, attention is called to the fact that, to-day, in the ordinary manufacturing plant operated by steam power, less than 5% of the total energy in the fuel consumed is available for useful work at the machine.

The curves in Figs. 1 and 2 show the great economy secured from seventy-five bituminous coals and six lignites when used in the gas producer instead of under the steam boiler. The results of the tests are officially reported on the basis of switch-board horse-power, but, in order that they may be of more practical value, they are given here on the basis of brake-horse-power and kilowatt—the efficiency of electric generator and belt in each plant being assumed as 85 per cent.

Especial attention should be called to the fact that several low-grade coals and lignites, which have proved of little value or even worthless under the steam boiler, have given excellent returns in the gas producer.

The ratios of the total fuel per brake-horse-power-hour required by the steam plant and the producer-gas plant under full-load conditions, not counting stand-by losses, as derived from 75 coals, 6 lignites, and 1 peat (Florida), are presented in Table 2.

TABLE 2.—Ratios of Fuels as Fired per Brake-Horse-Power-Hour under Boiler to Fuel as Fired per Brake-Horse-Power-Hour in Producer.

	Coals.	Lignites.	Peat.
Average	2.7	2.7	2.3
Maximum	3.7	2.9	....
Minimum	1.8	2.2	....

In the case of the results for the producer-gas tests, the figures include not only the coal consumed in the gas generator, but also

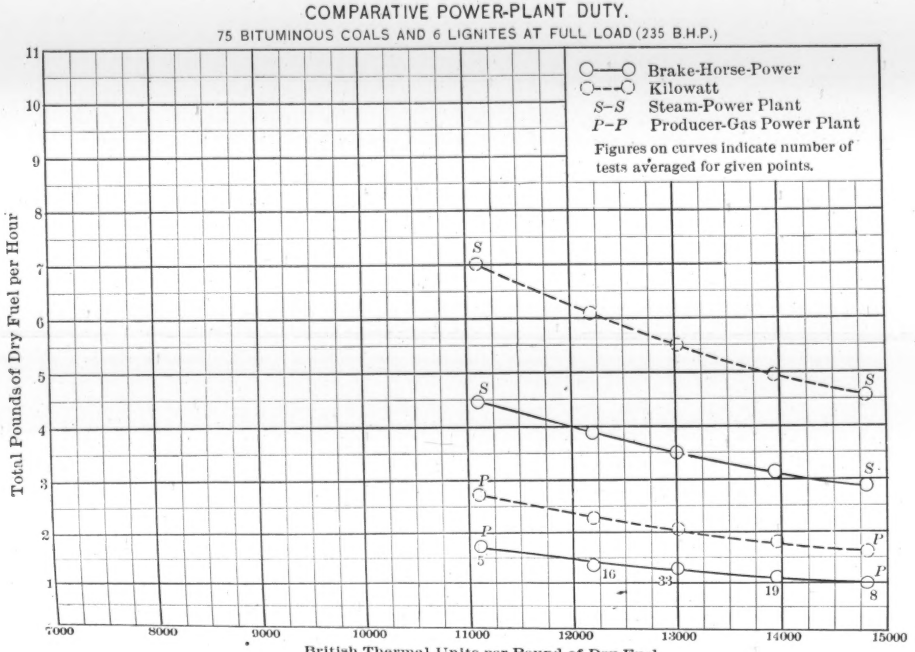

COMPARATIVE POWER-PLANT DUTY.
75 BITUMINOUS COALS AND 6 LIGNITES AT FULL LOAD (235 B.H.P.)
Brake-Horse-Power
Kilowatt
S–S Steam-Power Plant
P–P Producer-Gas Power Plant
Figures on curves indicate number of tests averaged for given points.
Total Pounds of Dry Fuel per Hour
British Thermal Units per Pound of Dry Fuel
7000
8000
9000
10000
11000
12000
13000
14000
15000
1
2
3
4
5
6
7
8
9
10
11
S
P
5
16
33
19
8

FIG. 2.

Mr. Fernald, the coal used in the auxiliary boiler for generating the steam necessary for the pressure blast, that is, the figures given include the total coal required by the gas-producer plant.

The relation between the coal required by the steam and producer-gas plant may be seen more clearly by an examination of Fig. 3, which presents graphically the results from thirty Illinois bituminous coals.

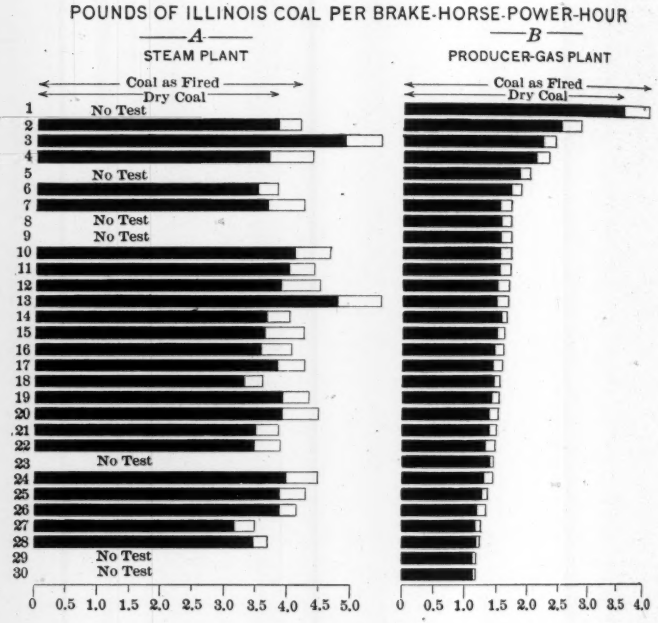


FIG. 3.

In considering the possible increase in efficiency of the boiler tests with a compound engine substituted for the simple engine used, the fact should not be overlooked that a corresponding increase in the efficiency of the producer-gas tests may be brought about under corresponding favorable conditions.

Too much emphasis cannot be given to the fact that all the tests upon bituminous coals, lignites, and peats were made in a

producer designed primarily for anthracite coal. It is, in fact, marvelous that any single plant can handle such a range of fuels. Not only is the producer passing through a transitional period, but the gas engine must still be regarded in the same light. In the larger sizes, the vertical, single-acting engine is being replaced by the horizontal double-acting type. Other changes and improvements are constantly being made which tend to do for the efficiency of the gas engine what compounding and tripling the expansions have already done for the steam engine. Mr. Fernald.

The engine used in the tests reported is, as already stated, of a type that is rapidly becoming obsolete for this size, namely, vertical, three-cylinder, single-acting.

A brief consideration of these points will lead at once to the conclusion that the producer-gas plant and steam plant used in these tests compare very favorably, and that any increase in efficiency in the boiler tests which might result from using a compound engine can be offset by the introduction of the more modern type of gas engine and a producer plant designed to handle the types of fuel used.

It should again be noted that many fuels are not fit to use under boilers. Many of these poor fuels have been used with the greatest ease in the gas producer, thus opening the way for the utilization of many fuels which have heretofore been regarded as practically of no value. Several of the poorest grades of bituminous coals have shown remarkable efficiency, and the lignites and peats have also responded with great readiness to the demands of the gas producer, thus opening the way to the introduction of cheap power into large districts which, thus far, have been commercially unimportant owing to the lack of industrial opportunities.

TIMOLEON GEELEN, ESQ. (by letter).—The following discussion relates principally to the development of gas generators in Europe. Mr. Geelen.

Besides illuminating gas, which is too costly for heating purposes, two other kinds of gas have been in use in Europe for a long time, namely, Siemens gas, and water gas.

Siemens gas is produced by the incomplete burning of cheap coal in a high furnace, and consists principally of carbonic oxide and nitrogen.

1 kg. of carbon, by combustion into carbonic oxide (CO), produces 2 400 t. u. (4 320 B. t. u. per lb.).

1 kg. of carbon, by combustion into carbonic acid (CO_2), produces 8 000 t. u. (14 400 B. t. u. per lb.).

The highest possible efficiency practically obtainable, therefore, is

$$\frac{8\,000 - 2\,400}{8\,000} = 0.7.$$

Mr. Geelen. As the temperature of the Siemens furnace is very high, the gas is not exactly suitable for power purposes, as it needs to be cooled, and therefore the efficiency of the generator is small. Water gas seems to be more suitable. It is produced by passing steam through burning coal, and consists principally of carbonic oxide and hydrogen.

$$C + H_2O = CO + 2H.$$

12 kg. of carbon require for their combustion 16 kg. of oxygen or 18 kg. of water, thus corresponding to 1 kg. of carbon to 1.5 kg. of water.

1 kg. of carbon, by combustion into CO, produces 2 400 t. u. (4 320 B. t. u. per lb.).

1.5 kg. of water requires by decomposition $1.5 \times 3\,200 = 4\,800$ t. u. ($1.5 \times 5\,760 = 8\,640$ B. t. u. per 1.5 lb.).

This shows that a water-gas furnace does not produce heat, but, on the contrary, consumes heat. For this reason, such a furnace cannot work continuously, but must be blast warm after having cooled itself. For a regular service, at least two furnaces are necessary, and these have to work alternately. The reversion, which is necessary every few minutes, makes the service very incommodious.

Water gas consists of equal parts of carbonic oxide and hydrogen. The molecular volume of each of the two kinds of gas is 23.13. The formation heat of water is 58 100 t. u., of carbonic oxide, 29 000 t. u., and of carbonic acid, 96 960 t. u. The heating value of the pure water gas, therefore, will be:

$(58\,100 + 96\,960 - 29\,000) \div (2 \times 23.13) = 2\,730$ t. u. per cu. m. (306 B. t. u. per cu. ft.).

The reaction in the Siemens furnace is: $C + O = CO$. As this furnace does not receive oxygen, but atmospheric air, a quantity of nitrogen also enters the process, namely:

$$\frac{0.787}{0.213} \times \frac{1}{2} \times 23.13 = 42.71 \text{ cu. m.}$$

The calorific value of the gas, therefore, is:

$(96\,960 - 29\,000) \div (23.13 + 42.71) = 1\,030$ t. u. per cu. m. (115 B. t. u. per cu. ft.).

It seemed advisable, therefore, to make a combination of the two processes, and introduce a mixture of air and steam in the furnace. The necessary heat for making the water gas is then furnished by the formation of the Siemens gas, and therefore the furnace can work continuously, the reaction being:

$$C + O = CO; \quad C + H_2O = CO + 2H.$$

In addition to 23.13 cu. m. of hydrogen and 46.26 cu. m. of carbonic oxide, the gas will also contain 42.71 cu. m. of nitrogen, as already mentioned; therefore, the 112.1 cu. m. of gas will contain

41% of carbonic oxide, 21% of hydrogen and 38% of nitrogen. Its calorific value will be: Mr. Geelen.

$[58\,100 + 2\ (96\,960 - 29\,000)] \div 112.1 = 1\,700$ t. u. per cu. m. (190 B. t. u. per cu. ft.).

Dowson, an Englishman, was the first to introduce an arrangement to produce such mixed gas, about 1885. At first he used anthracite as fuel, which gave him a pure gas; later, he also used coke. Fig. 4 shows the general arrangement. The water is evaporated in the boiler, *A*. The superheated steam passes through the suction air blast, *a*, and the mixture of air and steam passes under the grate. The gas produced in the generator, *B*, passes the cleaner, *C*, then through the scrubber, *E*, and the saw-dust cleaner, *D*. The gas then passes into the receiver.

DOWSON'S GAS PRODUCER

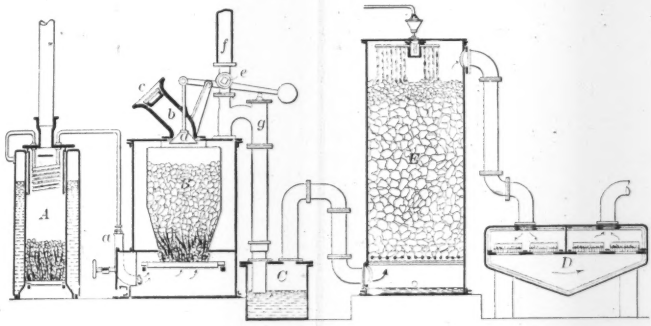


FIG. 4.

An attempt has been made to use the Dowson system by utilizing the heat of the gas which leaves the generator, and in this way an efficiency of 75% has been obtained. This means that the calorific value of the gas is equal to three-fourths of that of the carbon from which it has been produced.

The most important simplification in the construction of gas generators is that of Bénier, dating from about 1895. Instead of introducing the air by a steam blast, as in Dowson's arrangement, Bénier leaves the function of introducing the air to the engine itself, establishing in this way the well-known suction-gas arrangement. Fig. 5 shows the general arrangement of such an installation.

The evaporator forms a part of the generator, and the water is heated to a temperature of from 80 to 90° cent. by the passing gas.

Mr. Geelen. This evaporator is in communication with the atmospheric air through an opening, *e*. The entering air strikes the surface of the heated water and, after being saturated with water vapor, passes under the grate. The evaporator is provided with an overflow conducting the water into the ash-pit where it extinguishes the ash and also partly evaporates, thus contributing to the formation of gas.

A further change in the general construction of gas generators was made by Taylor, who separated the evaporator from the generator, resulting in the arrangement shown in Fig. 6.

Gas generators have also been constructed without the evaporator, the water being injected in small quantities which evaporate immediately. The advantage of this combination is that it is not

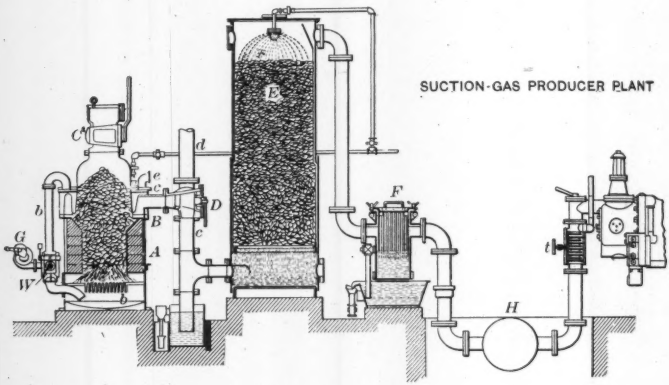


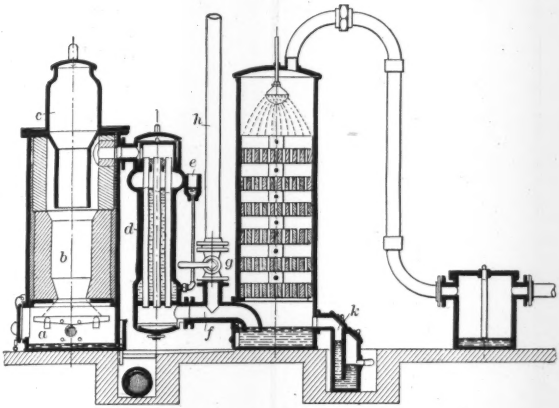
FIG. 5.

necessary to wait until the water in the evaporator has reached the required temperature before the machinery can be put into operation. Fig. 7 shows a generator of this kind.

The air enters the hollow cover of the generator at *a*, where it is heated, and then passes into the space, *b*, where it is heated still more by the gas. Into this space the water is injected and evaporates immediately, the mixture of air and water vapor being conducted under the grate through the pipe, *d*.

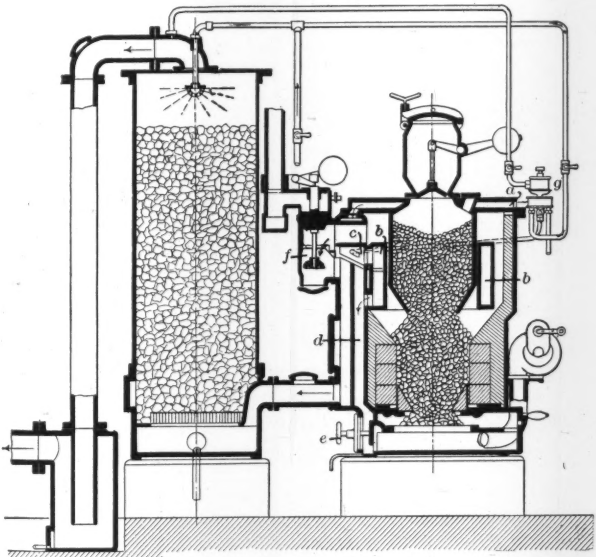
To adapt the quantity of injected water to the working suction of the engine, the generator is provided with a feed-water regulator, shown in Fig. 8. This regulator consists of an open receptacle into which water enters at *h*, and from which water flows to the evaporator through the valve, *i*. This valve is supported by the

Mr. Geelen.



SUCTION-GAS PRODUCER PLANT WITH SEPARATE EVAPORATOR

FIG. .



SUCTION-GAS PRODUCER PLANT WITH SEPARATE FEED-WATER REGULATOR

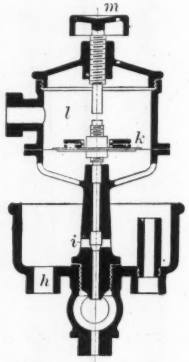
FIG. 7.

Mr. Geelen

flexible plate, *k*, which is the bottom of the box, *l*. The lift of this plate is limited by the screw, *m*. The box, *l*, is connected with the suction pipe of the engine. The stronger the suction, the higher the plate, *k*, will be lifted, and the more water will flow to the evaporator through the valve, *i*.

After leaving the generator the gas requires cleaning, and this is generally done in the different apparatus described hereafter.

At first the gas passes a receptacle filled with water, where it loses a portion of the dust it contains. Afterward it passes an apparatus generally called a scrubber. This is a high cylindrical receptacle filled with coke, the gas entering through an opening near the bottom. In passing upward it meets a spray of water



FEED-WATER REGULATOR

FIG. 8.

which cools it. In this apparatus the gas loses also a large portion of the tar it contains.

Another apparatus for cleaning the gas is the so-called condenser. It consists of a number of perforated plates placed in such a way that the gas passing through a hole of one plate strikes the unperforated part of the next plate, thereby losing a portion of the tar it contains.

If the fuel used contains much tar, the gas is forced to pass another cleaner, where it is purified on its way through a bed of saw-dust.

The installations which have been mentioned are only limited to the use of anthracite, coke or charcoal, having a consumption

of from 400 to 600 g. per h.p. per hour, according to the quality of the fuel. Mr. Geelen.

Fuel containing much tar, and also other cheap bituminous coal, cannot be used continuously with good results in these generators, and therefore constructors have been endeavoring to discover other arrangements better adapted to the use of such fuels.

The first to succeed in installing a large plant of this kind was Dr. Mond, in England. To avoid the baking of the fuel and the formation of clinkers, he introduced large quantities of steam into the generator, and burned the heavy tar-containing gas by forcing it to pass the hot fuel bed of the generator. The whole arrangement has a large calorific loss, and can only be executed on a large scale, because it cannot be worked with profit unless it is combined with an installation for the production of ammonia sulphate, the whole system being thus very complicated. Notwithstanding this inconvenience, the installation has given satisfactory results during ten years of service.

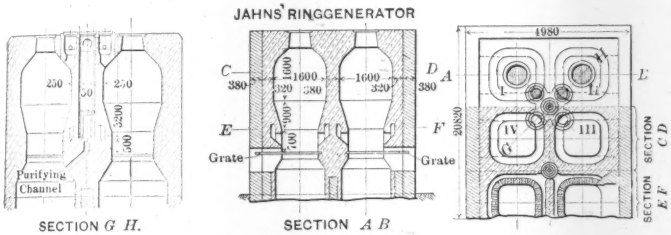


FIG. 9.

A new 12 000-h-p. plant, of the Duff design, has been installed recently in Madrid.

Quite a different system has given satisfactory results in Germany. This arrangement, called Jahn's ring generator, is working with the poorest coal, even with the so-called "Klaubeberge."

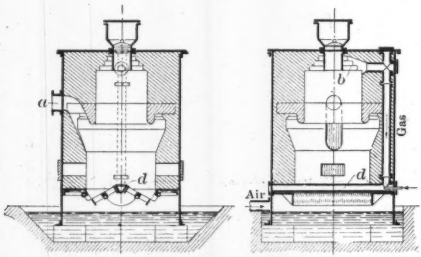
As seen from Fig. 9, Jahn's ring generator is composed of four generators working in one group. They are taken into service one after the other, and the gas of the third and fourth is led through the first and second, which are already burning well. In these generators is found a real formation of gas, and, besides this, the combustion and decomposition of the heavy tar-containing gas. When the fuel-contents of the first generator of the group, which has been longest in service, are no longer sufficient to aid in the process, this generator is put out of service, after being entirely exhausted. It is then cleaned and recharged, and is again connected to the other generators, the other three advancing correspondingly in the process.

Mr. Geelen. It must be stated, however, that this system, also, is only possible for large installations.

The modern construction of nearly all generators for the use of bituminous coal, lignite or turf, is based on the same principle, that of changing the fuel first into coke, and afterward gasifying this coke. The gas formed by the coking of the fuel contains much tar; this is decomposed, and the gas is made "permanent" by passing it through a bed of incandescent fuel in the generator.

The fuel column in these generators is made high, and the gas when produced leaves the generator in the middle of the column. The fuel in the upper part of the generator is coked, and the gas thus formed is led under the grate, being forced to pass the hot fuel bed of the generator, where it is made permanent. There is no loss in this process, as the carbonic acid and the water formed by the combustion are again reduced to carbonic oxide, hydrogen, and oxygen.

Fig. 10 shows the details of a generator of this kind.



DANIELS' GENERATOR
FIG. 10.

The gas is sucked off at *a;* the distillation gas produced in the upper part of the generator, *b,* falls through the pipe, *i,* into the pipe, *d,* the latter ending in the middle of the grate. Into this pipe steam is also introduced. The distillation gas is thus forced to pass the fire, and is made permanent.

A different arrangement to obtain the same effect is made by the construction of the so-called "double-working generators." They have two burning zones, one on the grate below, the other at the top of the fuel column, while the gas is sucked off in the middle of the column.

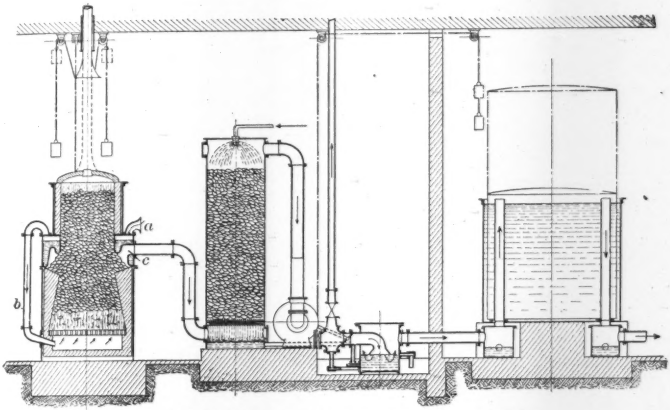
The fuel at the top of the column is at first coked, then passes into the upper burning zone, is extinguished on its way downward, and is finally consumed over the grate. The distillation gas formed in the upper part is sucked through the upper burning zone and is thereby made permanent.

Fig. 11 shows a type of these generators. Mr. Geelen.

It seems, however, that these generators also have not given entirely satisfactory results where coking coal is used, and it has been necessary to use efficient mechanical arrangements in their upper parts to prevent this baking.

One kind of bituminous coal has proved excellent for use in the double-working generators, this fuel being lignite.

Lignite is obtainable almost everywhere at a very low price. In 1902 the Gasmotorenfabrik Deutz, Germany, constructors of the Otto engines, had installed at the Düsseldorf Exhibition a suction-gas plant which worked with lignite. This installation is still in operation, and is giving entirely satisfactory results.



GENERATOR OF THE GASMOTORENFABRIK DEUTZ

Fig. 11.

In general, the suction-gas process, with lignite as fuel, is very simple. Lignite does not bake together, and the tar in the gas is not of as much importance as the tar in the gas from ordinary coal. The only inconvenience is the bad quality of the water leaving the cleaning apparatus.

In Germany lignite briquettes are used to a great extent, and have given excellent results. While the common lignite has from 40 to 60% of water and a calorific value of from 2 000 to 3 800 t. u. (from 3 600 to 6 840 B. t. u. per lb.), the lignite briquettes have less than 20% of water and a calorific value of about 4 500 t. u. (8 640 B. t. u. per lb.).

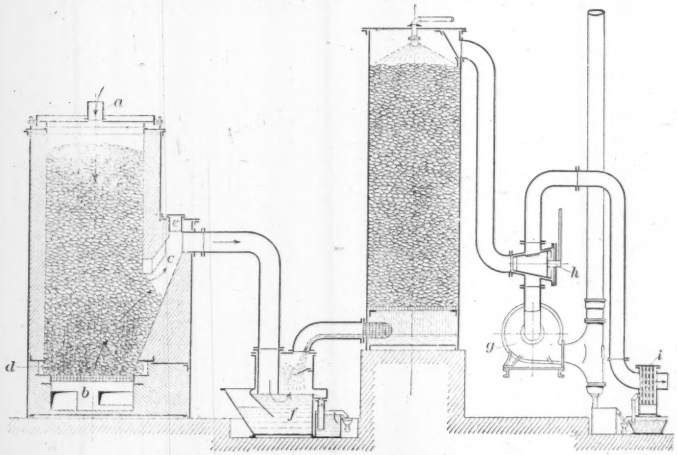
Fig. 12 is an illustration of a lignite generator.

Turf, also, can be used in the double-working generator; and even poor classes of turf, with from 40 to 50% of water, can be used

Mr. Geelen. advantageously. Turf gives from 1.3 to 1.8 cu. m. of gas per kg. (from 21 to 29 cu. ft. per lb.), corresponding to an average calorific value of from 1 350 to 1 499 t. u. per cu. m. (from 2 400 to 2 500 B. t. u. per cu. ft.).

Fig. 13 represents a generator for the use of turf.

The shaft consists of two parts, the upper and wider part being provided with two grates, and the lower and narrower part with the ordinary grate. During the service the turf burns only at the border of the upper grate. The inner part of the fuel column does not burn but sinks gradually downward, thereby being coked. The gas produced in the upper part of the generator passes through the hole, *c*, and the pipe, *d*, under the grate; then it goes through the burning zone, and is sucked off through the hole, *e*.



GENERATOR OF THE GASMOTORENFABRIK DEUTZ

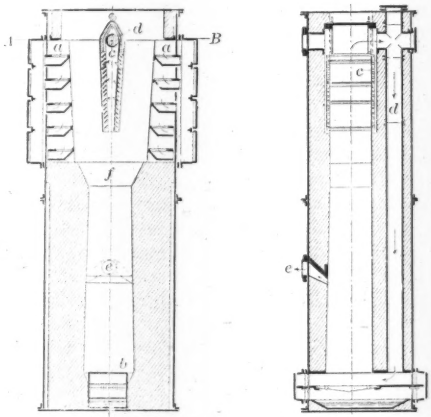
Fig. 12.

Another generator may be mentioned, namely, that of Riché, which is especially adapted for the gasification of wood. It must be stated, however, that wood alone does not give good results in service, and that a second fuel, carbon, is necessary to aid in the process. Fig. 14 shows the general arrangement of this generator.

It is composed of two shafts, the first serving as a feeding reservoir for the grate, *b*, on which the wood is burning, the second being filled with charcoal for the reduction of the produced carbonic acid. The tar-containing gas passes the fire, over the grate, and is thus burned and reduced to carbon oxide by the burning charcoal. The gas is then cooled in the pipes, *d* and *e*, by injected water.

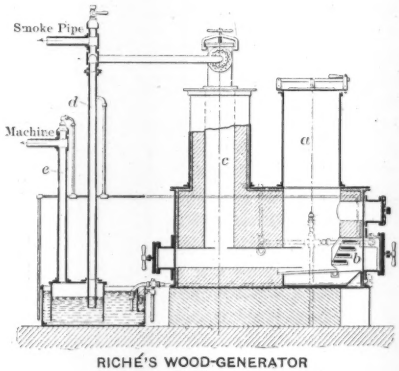
Mr. Geelen.

The idea of using two different kinds of fuel is also brought out in the generator of Lencauchez. However, instead of using two different generators, he uses only one for both kinds of fuel, as shown in Fig. 15.



KOERTING'S TURF-GENERATOR

FIG. 13.



RICHÉ'S WOOD-GENERATOR

FIG. 14.

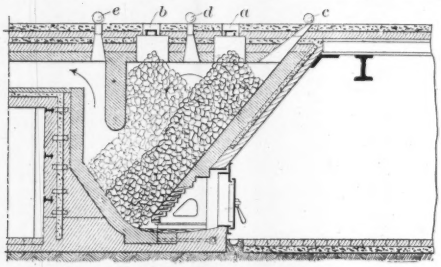
Gas coal is introduced into the generator through the hole, *a*, coke is introduced through the hole, *b*. The water is evaporated in the ash-pit. The gas produced is forced to pass the burning bed of coke, thus being reduced. The holes, *d* and *c*, will give black

Mr. Geelen. smoke, while the hole, *e*, is giving colorless gas. This generator is especially designed for heating purposes. From 75 to 80% of gas coal requires from 25 to 20% of coke.

The arrangements for cleaning the gas produced in the generators described are the same as those already mentioned in connection with the anthracite and coke generators.

It is not the writer's intention to give a full description of the great number of gas generators on the market. Nearly all these are based more or less on the principle already mentioned, namely, that of changing the fuel first into coke, which is then gasified, the tar-containing gas being forced to pass an incandescent bed of fuel, where it is made permanent.

The coke furnaces and high furnaces are generators of quite a different class. In Germany there are two different classes of coke furnaces. The first is the so-called "Flammöfen," which are heated



LENCAUCHEZ' GENERATOR

FIG. 15.

with all the gas the furnace produces. The heat of the burned gas, which has a temperature of 1 100° cent., is used everywhere for heating boilers. The result obtained therewith is, that for every kilogram of coal put into the furnace 1.2 kg. of water can be evaporated, thus regaining about one-sixth of the heat of the coal.

The second class of coke furnaces contains the so-called "Nebengewinnungsöfen," which deliver not only the hot burned gas but also coke gas. The gas leaving the coke furnace is first cooled, then it is separated from the tar, ammonia, benzol, etc., which it contains, and is then returned partially to the furnaces, where it heats them, the heat of the burned gas being utilized under steam boilers. The remaining gas could be utilized in gas engines, but it is also generally used to heat steam boilers. The quantity of this gas depends mainly on the quality of the fuel, and especially on the water it contains.

The combined heat of the burned gas and the coke gas of the "Nebengewinnungsöfen" does not evaporate as much water as the

heat of the burned gas of the "Flammöfen" alone, being only from 0.8 to 0.9 kg. of water per kg. of coal. Two-thirds of this belongs to the burned gas, and one-third to the remaining coke gas. Mr. Geelen.

If it is intended to dispose of the largest possible quantity of coke gas for use in gas engines, and thus not use the heat of the burned gas for heating steam boilers, the coke gas leaving the furnace is led into the so-called "Regeneratif" furnace. The quantity of gas which can be disposed of by using this furnace is from 100 to 140 cu. m. per ton of coal.

It is now a question of great interest to know how the highest possible horse-power can be obtained from the disposable heat. Experience has shown that nearly the same capacity is obtained in the three following cases:

1.—By utilizing the entire heat of the burned gas of the "Flammöfen" in modern steam boiler plants;

2.—By utilizing the heat of the burned gas of the "Nebengewinnungsöfen" in steam engines or turbines, and the remaining gas in gas engines;

3.—By utilizing the whole amount of gas of the regenerator furnace in gas engines.

At first sight it seems to be strange that, using a regenerator furnace and gas engines, it is not possible to produce a larger capacity. But the fact that a large amount of heat is lost through the chimney of the regenerator furnace has to be taken into consideration.

If only the "Nebengewinnungsöfen" are considered, about one-third more capacity can be obtained by installing a combined steam and gas plant, than by installing a whole steam plant; but the service is somewhat difficult with such a combined system, and will only give satisfactory results in large plants.

For this reason it must not be thought strange that, for instance in Germany, gas engines working with coke-furnace gas are not yet used to any great extent. If other arrangements are successful in obtaining more disposable gas from the regenerator furnaces, the coke gas engine will make its way as rapidly as the high-furnace gas engine.

In the high furnace the conditions are much more favorable, as it delivers considerable quantities of gas. However, in composition, coke-furnace gas and high-furnace gas are quite different. Coke-furnace gas, produced by the gasification of coal, contains principally methane (CH_4) and hydrogen, and has a very high calorific value of about 4 000 t. u. per cu. m. (448 B. t. u. per cu. ft.).

High-furnace gas, on the contrary, is a poor gas, and contains only one-third of combustible substances, especially carbonic oxide

Mr. Geelen. (CO), and has a calorific value of only from 800 to 900 t. u. per cu. m. (from 90 to 100 B. t. u. per cu. ft.).

High-furnace gas requires but little air in its combustion, about its own volume; coke-furnace gas requires from 5 to 6 times more air, and, notwithstanding this, the mixture is about 20% stronger than the high-furnace gas mixture, so that coke-gas engines could be built smaller than high-furnace gas engines. This is principally due to the hydrogen contents in coke-furnace gas.

Generally, however, the dimensions of each kind of machinery are made the same, taking into consideration an average indicated pressure of 4.8 atmospheres (about 70 lb.). The only difference in their construction is that the compression in the cylinder is different for the two kinds of gas, being for high-furnace gas at least 12 atmospheres (about 185 lb.), and for coke-furnace gas only 8 atmospheres (about 115 lb.).

The cleaning of these gases is of the greatest importance, and a distinction has to be made if the gas is to be used only for heating purposes or for gas-engine service. In the first case the dust contained in the gas may be about 0.5 g. per cu. m. (about 0.0003 lb. per cu. ft.), in the second case only a quantity of 0.02 g. per cu. m., or less, is admitted (about 0.0000013 lb. per cu. ft.).

However, as the better-cleaned gas has also given better results in steam-boiler plants, the same cleaning process is generally adopted at the present time for both cases of utilizing the gas.

The coke-furnace gas has its principal cleaning during the process of separating the tar, ammonia, benzol, etc., and has only to be cleaned of the remaining tar, sulphur and cyanide.

High-furnace gas has to be separated principally from the dust it contains.

The remaining tar in coke-furnace gas is separated by passing the gas through a coke-scrubber and afterward through a drying apparatus. Sulphur and cyanide are separated by passing the gas through an apparatus containing "Raseneisenerz," a carbonic iron ore. If the gas is not freed sufficiently from sulphur, it may soon be observed by the corrosion of the piston rod, valves, and other parts.

The separation of dust from high-furnace gas requires a considerable amount of force, being about 2% of the capacity of the working machinery. The general principle followed in this cleaning process is to force the dust into the water, with which it is then removed.

An apparatus of this type, very much used in Germany, is the centrifugal cleaner of Theisen, the construction of which is shown in Fig. 16. Besides this, a number of centrifugal cleaners have been put on the market, also a certain type of ball mill.

As already stated, the gas in these cleaning apparatus is freed from dust, so that it contains not more than 0.02 g. per cu. m. (0.0000013 lb. per cu. ft.). Therefore it contains less dust than the surrounding air in the working plant. As a final cleaner, a slack-wool, or a saw-dust, cleaner is used. Mr. Geelen.

The great importance of the use of coke-furnace gas and high-furnace gas is seen clearly, if an approximate calculation is made in reference to the capacity of coke and high furnaces.

During 1906 the production of coke in Germany was about 20 000 000 tons, corresponding to about 27 000 000 tons of coal.

For calculating the capacity which can be obtained from the disposable heat, we accept, per horse power per hour, in modern

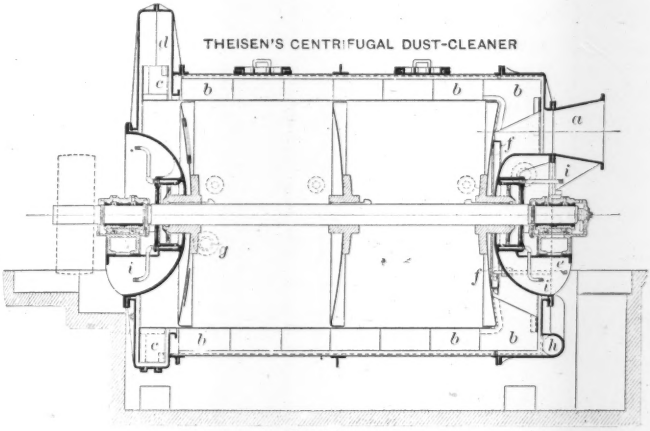


FIG. 16.

steam engines or turbines, 6 000 t. u. (about 1 500 B. t. u.) or 6 kg. (13.2 lb.) of superheated steam; and for modern gas engines, 3 000 t. u. (about 750 B. t. u.). In these figures all possible losses are included.

Taking this into consideration, we find for coke furnaces a disposable capacity of about 560 000 h-p. in combined steam engines and gas plants, or in pure gas plants.

Still more favorable conditions are found with high-furnace gas.

The iron production during 1906 in Germany was about 12 000 000 tons. Per ton of iron produced, 5 000 cu. m. of gas are produced (176 585 cu. ft.), with an average calorific value of about 800 t. u. per cu. m. (about 90 B. t. u. per cu. ft.). Of this quantity, 2 500 cu. m. are required to heat the air, thus leaving the remaining

Mr. Geelen. 2 500 cu. m. for use in gas engines. According to this, we find a disposal capacity of 1 000 000 h.p. in gas engines, or a capacity of 500 000 h.p. utilizing the gas in modern steam engines or turbines. Thus the total capacity which can be obtained in Germany alone by utilizing coke-furnace and high-furnace gas is about 1 500 000 h.p.

In the United States, where the production of coke and iron is much greater than in Germany, still more surprising figures may be found.

Coke-furnace and high-furnace gas received only special attention after the successful construction of large gas engines of a practical type. The construction of these engines was begun about 1890, and the makers encountered many difficulties. Nearly all constructors took the small gas engine (and the good results obtained with it) as a model for the construction of the larger types, and it was especially the construction of the cylinder head which caused the greatest difficulty; but, after abandoning this method of construction, and taking into consideration the results obtained in building large steam engines, they succeeded at last in making large gas-engine units, which would work under complete guaranty.

It is not the writer's intention to describe the very interesting details of the construction of these large gas engines. He will only state here that the largest gas engines yet built have a maximum capacity of from 1 000 to 1 500 h.p. per cylinder, and that the total maximum capacity of engines built in tandem-compound arrangement has been 3 500 h.p.

Mr. Buchanan. J. M. BUCHANAN, ESQ.—The speaker, in connection with Mr. Johnstone, has been for some time experimenting with a method of cleaning gas made in suction producers only. No attention, to speak of, has been devoted to any other kind of gas, but the results attained would indicate that at least some similar methods might be applied to blast furnace gas, or producer gas made in a different manner. Conditions in Mexico are such that clean fuel cannot always be obtained and one has to use what is nearest at hand, usually wood, peat, or lignite, sometimes rubbish, straw, pulque plant, in fact, anything of a burnable nature. Except when good anthracite coal or well-burned charcoal is used, it has been found that in all suction producers there is more or less sticky, gummy matter which passes off with the gas. If this matter is allowed to reach the engine, it causes clogging of the valves and sparkers, and it was soon found that the ordinary coke scrubber, using cobble-stones, wooden slats, etc., with water flowing over them, did not take out all this substance. It was also found that the more intimately the scrubbing water and the gas could be mixed, the more effective would be the separation of impurities, and, in the speaker's practice—and he believes it to be recommendable—the clearing of gas has been divided into three parts: namely, wet

scrubbing or washing; dry scrubbing or filtering; and centrifugal separation. Mr. Buchanan.

In the wet scrubbing process the gas is made to pass swiftly through a series of vertical pipes. There is a sprinkler giving a generous flow of water in the top of each pipe; and in each there is a series of baffle-plates shaped like two-bladed propellers—without the screw effect—placed about 2 in. apart, the full length of the pipe, and each set 60° from its neighbor, so that the whole forms a broken spiral. When the gas leaves the producer, it meets the stream of water coming down, its forced speed giving it sufficient action to turn the water into spray, and the two (gas and water) mix thoroughly. At the bottom of the first pipe there is a return bend to let the water get away. There is also an outlet pipe at the lowest point, but, as there is a seal or trap, the gas cannot escape nor can air enter. The gas going up then meets another down-coming stream, and runs against the water through the second series of baffle-plates, then around the second return bend, and down over a third series of baffle-plates, the water, of course, going down with it. The gas then passes to the dry scrubber where the water separates and is drawn off. The principle involved is that the gas, flowing at high speed past and around the baffle-plates, whips the water into such fine particles and comes into contact with it so intimately that the impurities are caught, entrained, and carried off in the water. Matter such as lamp black, soot, and other substances too fine to be caught by the water will lodge in a dry scrubber having two or more layers of straw, well oiled with crude petroleum or a similar oil, and packed tightly—the more tightly packed, the better is the cleansing effect. Excelsior can be used, also wild grass, or anything that will pack tightly and yet allow the gas to pass through. This straw, where the gas first reaches it, should be removed occasionally, and, as it makes good gas fuel, it can go into the producer and be gotten rid of at a profit, which is in itself an advantage. Again, the more tightly the straw is packed, the sooner it will choke; this, though, saves the engine, and it is manifestly easier and better to change the straw in the scrubber than to clean a dirty engine, not to speak of the safer and steadier operation.

With fuels that give off matter which is too refractory to handle by these methods alone, centrifugal separation is used, presumably similar to that mentioned by Mr. Geelen; gas and water come down through the pipe emulsionizer, mix together, and pass into the centrifugal separator (which is built much like a centrifugal pump, in four stages or more) previous to entering the dry scrubber. By this process everything that water will catch or precipitate is taken out, leaving the non-precipitatable carbonic oxide, hydrogen, and other gases clean. The dry scrubber then has almost no work to do, and the gas reaches the engine in a clean and dry condition. The sub-

Mr. Buchanan. stances taken out by the series of pipes and baffle-plates (which has been called the "emulsionizer") and the centrifugal separator, sometimes have and sometimes have not a market value, depending on the nature of the fuel burned in the producer. The principle, it is believed, is different from that usually followed, and can be carried out efficiently with gas of any kind as well as suction producer gas.

When a suction producer (which is the only type which the speaker cares to discuss definitely) is first started, the fuel bed is thin and the gas somewhat lean; as the producer heats up, owing to long-continued running, the fuel bed becomes thicker, and the gas varies considerably, and usually the mixture has to be changed two or three times, or as many as five or six times, before it will settle down to a steady run. By the "mixture" is meant the portions of air and gas necessary to make the proper expulsion.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 1061.

COMPARISON BETWEEN RAINFALL AND RUN-OFF IN THE NORTHEASTERN UNITED STATES.*

By John C. Hoyt, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. E. C. Murphy, Charles A. Holden, H. C. Frankenfield and Alfred J. Henry, Thaddeus Merriman, Raphael Zon, C. C. Vermeule, H. K. Barrows, J. C. Stevens, M. O. Leighton, F. H. Newell, A. P. Davis, N. C. Grover, F. W. Hanna, and John C. Hoyt.

Until within a comparatively recent date it has been the common practice of engineers to estimate the discharge of surface streams from precipitation data. This practice was necessary because of the comparatively few reliable data in regard to stream flow, and, although responsible for many wild estimates, it has served a useful purpose and has become so common that many engineers have persisted in its use even when reliable run-off data were available.

During the past ten years a large amount of information has been collected by the United States Government and by private parties, in regard both to precipitation and to stream run-off, which makes it now possible to compare in detail the relation between these two phenomena. Such a study, for the northeastern portion of the United States, is herein presented.

The tables† used for the comparison show, not only the relation of the rainfall to the run-off, but have also the additional practical value of serving as a basis for predicting the average run-off which may be expected in the areas which have been investigated.

* Presented at the meeting of September 4th, 1907.

† Acknowledgment is made to Mr. Fred F. Henshaw for his assistance in the preparation of the tables.

At the start it was intended to extend the comparisons to all sections of the United States where run-off data are available. The time required for such an extensive study would have been so great, however, that it has been decided to present the data now in hand, covering only the northeastern portion of the United States.

Rainfall Data.

The rainfall data have been collected by the United States Weather Bureau. In using them, the following conditions must be considered:

The plan of the Weather Bureau is to establish a precipitation station in each county; therefore, in most of the areas considered, the horizontal distribution of the stations is uniform and quite representative. This is not generally the case, however, with the vertical distribution of the stations, as the location of the gauge depends in a large measure on the accessibility of a reliable observer and a telegraph station, and these are not generally available at the higher altitudes. Most of the stations have been located at low or medium elevations; and, as but few definite data are available on the effects of elevation on precipitation, it has been impracticable in this study to allow for this factor.

A rain gauge collects the precipitation that falls on a few square inches of surface; therefore, in order to measure accurately the average precipitation over a large area, there should be many rain gauges. As the gauges are always few in number, an extreme precipitation occurring on a comparatively small area may not be recorded.

No satisfactory method has yet been devised for obtaining reliable data in regard to precipitation in the form of snow, and practically no data have been collected to show the quantity of water stored as snow at the end of each month. There is introduced, therefore, an uncertainty in the winter records which makes it necessary to study the winter period as a whole.

It is assumed that for any month the mean rainfall over the whole area is the mean of the monthly rainfalls at the various stations in that area. Based upon this assumption, the monthly and yearly rainfalls have been determined for each of the years when run-off records were available.

Records of precipitation were lacking for many months in these years. In all such cases the mean of the available records was taken as the mean for the month in question; therefore, it may happen that the mean for January may be obtained from records of 20 stations, while the mean for February may come from only 15 stations. In Table 2 the last column, "Number of precipitation stations," shows the least number of stations used for any month of the year.

Run-off Data.

The run-off data used in this study have been taken from the records of the United States Geological Survey. The data, in all cases, have been reviewed and brought to a uniform basis, special attention being given to the winter estimates.

The run-off of a stream results from precipitation, and is the water left after evaporation, vegetation, seepage, etc., have been satisfied, plus the flow from ground-water. The seasonal distribution and rate of run-off depend mainly upon climate, topography, geology, vegetation, and ground, snow, or surface storage. Owing to lack of detailed information in regard to these regulating factors no attempt has been made to discuss the magnitude of their effects.

Comparison of Rainfall and Run-off.

In order to compare rainfall and run-off, the data have been expressed as "depth in inches," over the drainage basins in question, for uniform periods of time, usually the calendar month. This period, however, is too short for comparison, and leads to results which are apparently erroneous, because heavy precipitation may occur at the end of the month and not appear as run-off until the following month.

The year is a better period for the comparison of these quantities, but is not entirely satisfactory. The calendar year is unsatisfactory because the conditions of snow and ground storage are not the same at the end of each calendar year. A year beginning with October 1st makes a much better period, because on that day conditions are much more nearly uniform for successive years, as there is no snow storage, the ground-water will have been lowered to about the same level each year, and the ground storage is gen-

erally a minimum. This period, therefore, has been used in the following comparisons.

Areas Considered in the Comparisons.

The areas used in this discussion are indicated in Table 1 and shown on Plate LIX. These, with the exception of the Ohio above Wheeling, drain into the North Atlantic Ocean, and are representative of the New England States, New York, New Jersey, Pennsylvania, Maryland, Virginia and West Virginia.

The stations lowest down in the drainage basin, with the exception of the Roanoke drainage, have been designated "principal stations," and those in the upper portions of the area, "secondary stations."

TABLE 1.—List of Areas.

River.	Drainage above:	Percentage of accuracy.	Number of gauge readings daily.	Drainage area, in square miles.	Length of records, in years.
Connecticut	Orford, N. H.	5 (ice 10)	1	3000	5
Housatonic	Gaylordsville, Conn.	5	2	1 020	5
Susquehanna	Harrisburg, Pa.	5 to 10	1	24 030	14
Susquehanna	Wilkes-Barre, Pa.*	5 to 10	1	9 810	6
W. Br. Susquehanna	Williamsport, Pa.*	5 to 10	1	5 640	10
Ohio	Wheeling, W. Va.	5	1	23 820	21
Potomac	Point of Rocks, Md.	5 to 10	1	9 650	10
Shenandoah	Millville, W. Va.*	5 to 10	1	3 000	10
James	Cartersville, Va.	5 to 10	1	6 232	7
James	Buchanan, Va.*	5 to 10	2	2 060	10
James	Glasgow, Va.*	5 to 20	1	830	10
Appomattox	Mattoax, Va.*	5 to 15	2	745	5
Roanoke	Roanoke, Va.	5 to 10	1	390	9
Roanoke	Randolph, Va.*	5 to 15	1	3 080	5

* Secondary stations.

Description of Tables.

For comparative purposes, the following tables have been prepared:

Table 1.—List of areas.

Table 2.—Monthly rainfall, run-off, run-off in percentage of rainfall, and loss, over the area above each gauging station.

Table 3.—Monthly and yearly maximum, minimum and mean rainfall, run-off, run-off in percentage of rainfall, and loss for an average year over the area above each gauging station.

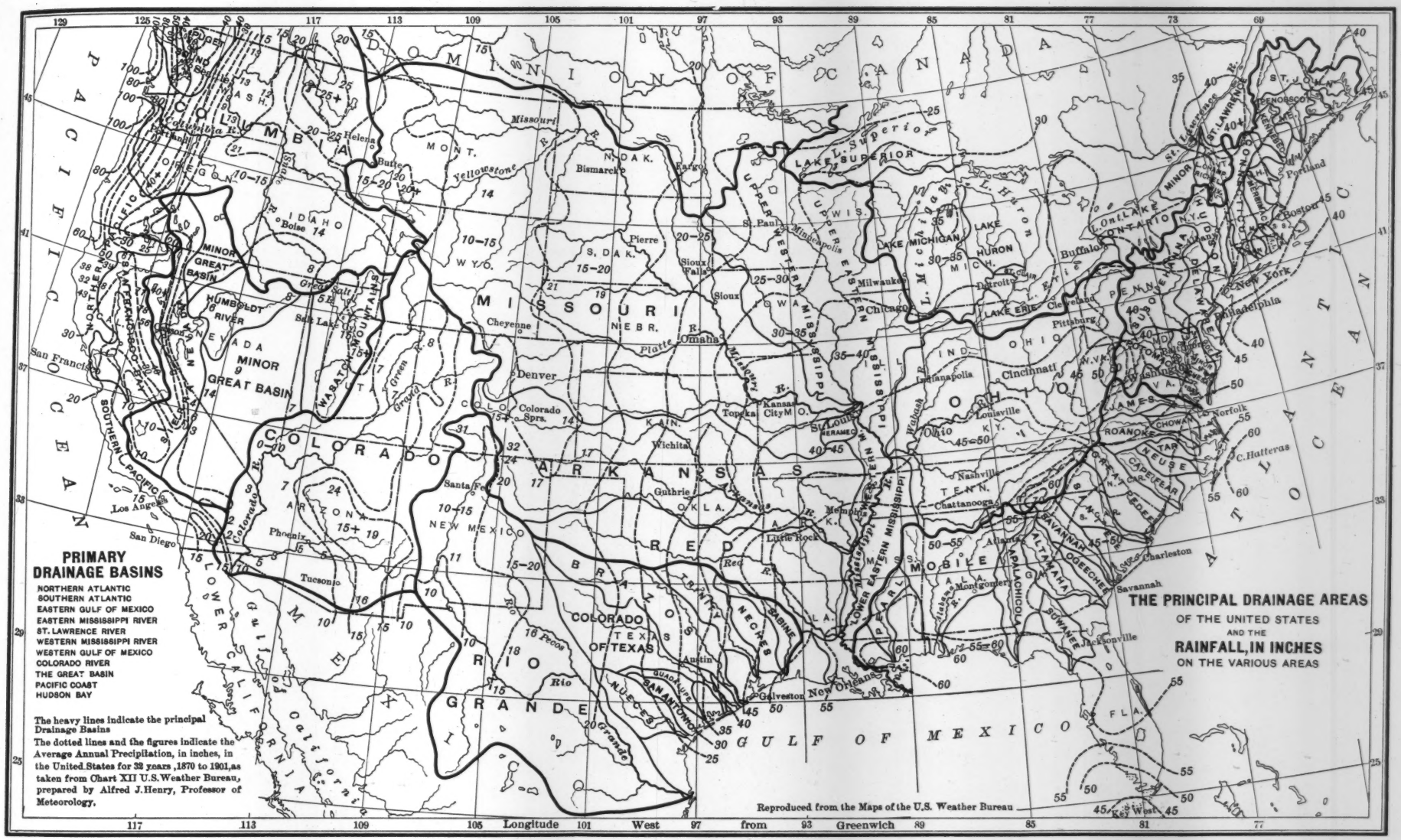

PRIMARY
DRAINAGE BASINS
NORTHERN ATLANTIC
SOUTHERN ATLANTIC
EASTERN GULF OF MEXICO
EASTERN MISSISSIPPI RIVER
ST. LAWRENCE RIVER
WESTERN MISSISSIPPI RIVER
WESTERN GULF OF MEXICO
COLORADO RIVER
THE GREAT BASIN
PACIFIC COAST
HUDSON BAY
The heavy lines indicate the principal Drainage Basins
The dotted lines and the figures indicate the Average Annual Precipitation, in inches, in the United States for 32 years, 1870 to 1901, as taken from Chart XII U.S. Weather Bureau, prepared by Alfred J. Henry, Professor of Meteorology.
THE PRINCIPAL DRAINAGE AREAS
OF THE UNITED STATES
AND THE
RAINFALL, IN INCHES
ON THE VARIOUS AREAS
Reproduced from the Maps of the U.S. Weather Bureau
Longitude West from Greenwich
PACIFIC OCEAN
ATLANTIC OCEAN
GULF OF MEXICO
DOMINION OF CANADA
MISSOURI
COLORADO
ARKANSAS
RED
BRAZOS
RIO GRANDE
MOBILE
GREAT BASIN
MINOR GREAT BASIN
COLORADO OF TEXAS
LAKE SUPERIOR
LAKE MICHIGAN
LAKE ERIE
ONTARIO
MINOR
HURON
Columbia R.
Missouri R.
Mississippi R.
Ohio R.
Rio Grande
Colorado R.
Red R.
Arkansas R.
Yellowstone
Platte R.
Kansas R.
Snake R.
Green R.
Gila R.
Pecos
Brazos
Trinity
Neches
Sabine
Pearl
Alabama
Tombigbee
Apalachicola
Savannah
Ogeechee
Roanoke
James
Potomac
Susquehanna
Delaware
Hudson
Connecticut
Kennebec
Penobscot
St. John
St. Lawrence
Merrimac
Portland
Boston
New York
Philadelphia
Pittsburg
Cleveland
Buffalo
Detroit
Milwaukee
Chicago
Minneapolis
St. Paul
Duluth
Bismarck
Fargo
Pierre
Sioux Falls
Omaha
Cheyenne
Denver
Colorado Sprs.
Kansas City
Topeka
St. Louis
Wichita
Guthrie
Indianapolis
Cincinnati
Louisville
Nashville
Memphis
Little Rock
Chattanooga
Atlanta
Montgomery
Jacksonville
Charleston
Norfolk
Cape Fear
Hatteras
Key West
Galveston
New Orleans
Austin
San Antonio
Houston
Santa Fe
Phoenix
Tucson
Los Angeles
San Diego
San Francisco
Portland
Boise
Butte
Helena
Salt Lake City
Great Salt L.
Humboldt River
Truckee R.
Carson R.
Sacramento R.
Colorado R. of California
Gulf of California
Lower California
Mexico
Idaho
Wyo.
Mont.
N. Dak.
S. Dak.
Nebr.
Kans.
Okla.
Ark.
Tex.
New Mexico
Ariz.
Nev.
Utah
Colo.
Wash.
Ore.
Calif.
Ill.
Ind.
Ohio
Ky.
Tenn.
Miss.
Ala.
Fla.
Ga.
S.C.
N.C.
Va.
W.Va.
Pa.
N.Y.
Md.
Del.
N.J.
Conn.
R.I.
Mass.
Vt.
N.H.
Me.
Mich.
Wis.
Minn.
Iowa
Mo.

Table 4.—Yearly rainfall, run-off, run-off in percentage of rainfall, and loss, over the Susquehanna, Potomac, James and Roanoke basins.

Table 5.—Yearly rainfall, run-off, run-off in percentage of rainfall, and loss, over the area above each principal station.

Table 6.—The winter rainfall, run-off, run-off in percentage of rainfall, and loss, as indicated by the months of December, January, February, March and April; and the growing period of rainfall, run-off, run-off in percentage of rainfall, and loss, as indicated by the months of June, July and August.

General Discussion.

The drainage basins considered may be grouped as follows:

First.—High latitudes, as represented by the Connecticut above Orford, N. H.

Second.—Coast area, as represented by the Housatonic at Gaylordsville, Conn.

Third.—Medium latitude areas, as indicated by the Ohio at Wheeling, W. Va., and the Susquehanna at Harrisburg, Pa.

Fourth.—Mountain-surrounded area, as indicated by the Potomac above Point of Rocks, Md.

Fifth.—Southern areas, as shown by the James above Cartersville, Va., and the Roanoke above Roanoke, Va.

The general conditions of rainfall and run-off in the various basins are shown in Table 3, which gives the mean monthly and yearly values for those basins.

Neglecting the data for the Housatonic and Potomac drainages, which are in special classes, Table 3 shows a gradually increasing rainfall from the north to the south, varying from 36.76 in. in the Connecticut drainage to 42.68 in. in the Roanoke drainage. The large rainfall above Gaylordsville (47.86 in.) is explained by its proximity to the coast line, where the evaporation, and therefore the rainfall, is much greater than in the interior. The low rainfall in the Potomac basin (36.86 in.) may be accounted for by the topography of this basin, a large portion of which is surrounded by

high mountains which precipitate a large percentage of the moisture from the rain clouds, and thus reduce the rainfall beyond them.

The run-off is very consistent in the various groups, and decreases toward the south, although the rainfall increases. It is about 60% of the precipitation in the northern areas, 55% in the intermediate areas and 40% in the southern areas. This decrease in run-off is due to the increase in evaporation and the loss by vegetation, and shows that the climate and vegetation are probably the principal regulating factors in the relation between rainfall and run-off.

The actual loss of rainfall increases quite rapidly toward the south, ranging from 15.10 in. in the Connecticut drainage to 24.99 in. in the Roanoke drainage; the mean loss for frozen areas being 17.7 in. and for southern areas 24.13 in.

Table 4 gives the relative conditions in the various parts of the same drainage basins, and shows no perceptible difference in conditions between the various branches and the main stream. Unfortunately, data are not available for the extreme upper portions of the drainages, where it is probable that the run-off is somewhat greater than in the lower portions.

Table 5 and Fig. 1 show the relation between years of maximum and minimum rainfall on run-off and loss, as follows:

1.—High rainfall produces a correspondingly high run-off.

2.—Low rainfall produces a correspondingly low run-off.

3.—The run-off in percentage of rainfall varies with the rainfall. It is likely to be high during the years following high rainfall and low during the years following low rainfall.

4.—Small apparent loss occurs in years showing large rainfall.

5.—Large apparent loss occurs after years of small rainfall.

These peculiarities are due mainly to the effect of ground storage, which is the most important factor governing run-off. At the ends of years of high precipitation the ground-water table will be high. Maximum ground storage exists at the end of maximum precipitation. The quantity of stored water carried into the following year will be large unless the rainfall has been small for the last

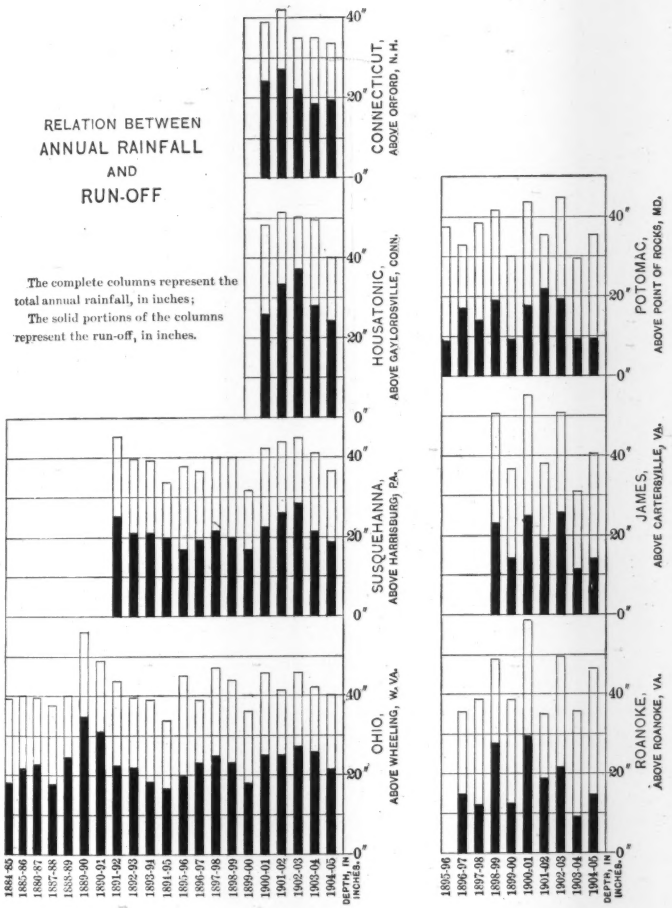

RELATION BETWEEN
ANNUAL RAINFALL
AND
RUN-OFF
The complete columns represent the total annual rainfall, in inches;
The solid portions of the columns represent the run-off, in inches.
CONNECTICUT, ABOVE ORFORD, N.H.
HOUSATONIC, ABOVE GAYLORDSVILLE, CONN.
SUSQUEHANNA, ABOVE HARRISBURG, PA.
OHIO, ABOVE WHEELING, W. VA.
POTOMAC, ABOVE POINT OF ROCKS, MD.
JAMES, ABOVE CARTERSVILLE, VA.
ROANOKE, ABOVE ROANOKE, VA.
0″
20″
40″
DEPTH, IN INCHES.
1884-85
1885-86
1886-87
1887-88
1888-89
1889-90
1890-91
1891-92
1892-93
1893-94
1894-95
1895-96
1896-97
1897-98
1898-99
1899-00
1900-01
1901-02
1902-03
1903-04
1904-05
1905-06

FIG. 1.

month or two, thus allowing the excess to drain off. Exceptions to the above are readily explained by extreme conditions during the latter months of the year, as shown in Table 2.

Table 6 shows the relation during the winter months, December to April, inclusive, and the summer or growing months, June to August, inclusive.

A study of these tables shows that in the northern area, where the ground is frozen, and where there is considerable snow during the winter months, the run-off very nearly equals the precipitation, being for the principal basins 92%, which shows that the evaporation and other loss, neglecting any gain or loss from ground storage, is very small during the winter period.

For the southern drainages, where the ground is frozen for a shorter period, and is rarely covered with a great depth of snow, the run-off during the winter months averages 63% for the principal basins.

That the climatic conditions are responsible for this change in the percentage of run-off between northern and southern areas, rather than geologic conditions or topography, is shown in the "percentage" column for the summer months, where the percentage of run-off varies between 20 and 32, having a mean of 27% for all the principal basins. This column shows no regular variation for the basins. It also shows that the evaporation and loss through evaporation is very nearly the same over the various areas considered, being about 9.5 in.

These relations for the winter months give a method of checking in a general way the winter run-off of streams in frozen areas, by comparison with the total precipitation for the winter months. Although this check is only approximate, it may be of considerable value, as the records of winter flow, as a rule, are very meager and unsatisfactory, owing to uncertainties in regard to ice conditions.

Table 2 gives the general conditions of rainfall and run-off for the various drainages for the individual months, and forms a basis for the other tables. The wide range of variations in this table shows the impossibility of estimating run-off from rainfall data. These variations, however, as a rule, may be explained by the variation of ground-water conditions, although the exact ground-water conditions may not be known. For example, the table for the

Potomac at Point of Rocks shows, in October, 1896, 162% of run-off, while the mean run-off for that month is 24% for all years. That this value is not in error is shown by the precipitation in September, which was 6.09 in., while the normal for September is only 2.65 in. This high precipitation in September evidently left an abnormally high ground-water at the end of the month, which maintained the flow during the following month. It will be noted, also, that the percentage of run-off for September was abnormally low, being only 4%, while the precipitation was abnormally high. In August there was an abnormally low precipitation and a correspondingly low ground-water, which was replenished by the heavy rainfall of September. This is only one of the many examples which can be pointed out in the tables for the various basins.

The principal reason, therefore, why rainfall data fail, as a basis for estimating run-off, is the lack of knowledge of ground-water conditions. If data in regard to these conditions were available, estimates from reliable rainfall data could no doubt be made more satisfactorily.

Special attention is called to Table 3, which shows the average conditions of run-off over the various areas. This table is of especially practical value to the engineer in showing him the conditions which may be expected within the limits of the areas discussed or for similar areas. It gives him not only the mean conditions, but also the absolute maximum and minimum during the years in question. The comparison of drainage areas, in connection with such tables of run-off should be the only basis for estimating the run-off of streams on which no measurements of flow have been made.

Run-off, in Percentage of Precipitation.

1900-01.....	42	45	103	50	86	45	204	65	63	23	30	22	62	.....
1901-02.....	33	33	49	55	28	137	120	83	68	38	38	30	65	.....
1902-03.....	37	132	39	36	32	163	266	430	20	20	23	34	63	.....
1903-04.....	17	42	17	13	23	80	120	141	49	13	16	22	46	.....
1904-05.....	92	83	(35)	15	26	101	220	130	35	30	26	33	58	.....
Mean.....	41	57	46	33	34	114	170	104	45	25	28	28	59	.....
Maximum....	92	132	103	55	86	163	266	430	68	38	39	34	65	.....
Minimum....	17	33	17	13	23	45	120	65	20	13	16	22	46	.....

Loss, in Inches.

1900-01.....	2.10	3.03	−0.06	0.99	0.10	1.92	−3.62	1.67	1.05	3.81	2.00	1.94	14.93	.....
1901-02.....	1.61	1.20	2.43	0.91	1.36	−1.77	−0.72	0.74	1.51	2.41	2.45	2.63	14.76	.....
1902-03.....	2.66	−0.41	2.08	1.88	2.15	−2.91	−2.27	−0.89	4.23	3.19	2.48	0.71	12.84	.....
1903-04.....	2.24	0.70	2.29	1.87	0.92	0.41	−0.70	1.39	1.08	3.27	3.62	4.59	18.81	.....
1904-05.....	0.18	0.18	(0.88)	1.97	0.75	−0.02	2.37	−0.67	2.59	3.55	3.39	3.74	14.17	.....
Mean.....	1.75	0.94	1.52	1.52	1.06	−0.47	−1.94	−0.11	2.09	3.25	2.79	2.70	15.10	.....
Maximum....	2.60	3.03	2.43	1.97	2.15	1.92	−0.70	1.67	4.23	3.81	3.62	3.74	18.81	.....
Minimum....	0.18	−0.41	−0.06	0.91	0.10	−2.91	−3.62	−1.39	1.05	2.41	2.00	0.71	12.84	.....

TABLE 2.—MONTHLY RAINFALL, RUN-OFF, RUN-OFF IN PERCENTAGE OF RAINFALL, AND LOSS.
CONNECTICUT RIVER ABOVE ORFORD, N. H. DRAINAGE AREA, 3 300 SQ. MILES.

PRECIPITATION.

Year.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Period.	No. Prec. Sta.
1900-01.....	3.63	5.54	1.73	1.99	0.71	3.52	3.48	4.78	2.81	4.97	3.29	2.48	38.93	10
1901-02.....	2.39	1.78	4.77	2.02	1.89	4.77	3.54	4.27	4.71	3.90	3.98	3.78	41.80	10
1902-03.....	4.12	1.27	3.39	2.94	3.18	4.65	1.37	0.27	5.27	4.00	3.22	1.08	34.76	10
1903-04.....	2.69	1.20	2.76	2.14	1.19	2.05	3.46	3.41	2.10	3.76	4.31	5.75	34.82	11
1904-05.....	2.12	1.05	1.36	2.32	1.01	2.20	1.98	2.23	4.00	5.06	4.59	5.56	33.48	8
Mean.....	2.99	2.17	2.80	2.28	1.60	3.44	2.77	2.99	3.78	4.34	3.88	3.73	36.76	.....
Maximum....	4.12	5.54	4.77	2.94	3.18	4.77	3.54	4.78	5.27	5.06	4.59	5.75	41.80	.....
Minimum.....	2.12	1.05	1.36	1.99	0.71	2.05	1.37	0.27	2.10	3.76	3.22	1.08	33.48	.....

RUN-OFF.

1900-01.....	1.53	2.51	1.79	1.00	0.61	1.60	7.10	3.11	1.76	1.16	1.29	0.54	24.00	.....
1901-02.....	0.78	0.58	2.34	1.11	0.53	6.54	4.26	3.53	3.20	1.49	1.53	1.15	27.04	.....
1902-03.....	1.52	1.68	1.31	1.06	1.03	7.56	3.64	1.16	1.04	0.81	0.74	0.37	21.92	.....
1903-04.....	0.45	0.50	0.47	0.27	0.27	1.64	4.16	4.80	1.02	0.49	0.69	1.25	16.01	.....
1904-05.....	1.94	0.87	(0.48)	0.35	0.26	2.22	4.35	2.90	1.41	1.51	1.20	1.82	19.31	.....
Mean.....	1.24	1.23	1.28	0.76	0.54	3.91	4.70	3.10	1.69	1.09	1.09	1.03	21.66	.....
Maximum....	1.94	2.51	2.34	1.11	1.03	7.56	7.10	4.80	3.20	1.51	1.53	1.82	27.04	.....
Minimum....	0.45	0.50	0.47	0.27	0.26	1.60	3.64	1.16	1.02	0.49	0.69	0.37	16.01	.....

Run-off, in Percentage of Precipitation.

1900-01.....	(13)	22	65	59	64	87	105	72	113	15	22	29	53	.....
1901-02.....	64	50	61	149	(39)	203	105	80	27	30	40	17	64	.....
1902-03.....	51	175	67	92	(87)	151	170	99	59	61	31	80	73	.....
1903-04.....	50	78	67	(41)	(48)	(129)	109	77	48	37	20	23	56	.....
1904-05.....	58	69	(37)	(63)	(84)	(160)	187	118	20	11	18	46	60	.....
Mean.....	47	58	61	76	63	143	125	81	41	29	25	32	62	.....
Maximum....	64	175	67	149	(87)	203	187	118	113	61	40	80	73	.....
Minimum	(13)	22	(37)	(41)	(39)	87	105	72	20	11	18	17	53	.....

Loss, in Inches.

1900-01.....	(2.70)	3.33	0.96	0.67	0.27	0.66	—0.31	1.74	—0.25	4.25	5.71	2.83	22.56	.....
1901-02.....	0.98	1.20	2.63	—1.05	(2.36)	—4.46	—0.20	0.60	3.93	5.06	2.08	5.19	18.32	.....
1902-03.....	2.31	—0.67	1.93	0.26	(0.56)	—2.35	—1.57	0.01	6.14	1.56	4.82	0.39	13.39	.....
1903-04.....	3.24	0.54	1.25	(1.99)	(1.38)	(—0.95)	—0.36	0.76	2.60	2.36	4.14	1.92	21.87	.....
1904-05.....	1.20	0.59	(1.73)	(1.82)	(0.20)	(—1.81)	—2.20	—0.22	—3.67	4.41	4.00	2.62	16.01	.....
Mean.....	2.09	1.00	1.70	0.73	0.95	—1.78	—0.92	0.57	3.22	3.53	4.15	3.19	18.43	.....
Maximum....	3.24	3.33	2.63	(1.99)	2.36	0.66	—0.20	1.74	6.14	5.06	5.71	5.19	22.56	.....
Minimum....	0.98	—0.67	0.96	—1.05	(0.20)	—4.46	—2.20	—0.22	—0.25	1.56	2.08	0.39	13.39	.....

TABLE 2—(*Continued*).

HOUSATONIC RIVER, ABOVE GAYLORDSVILLE, CONN. DRAINAGE AREA, 1 020 SQ. MILES.

PRECIPITATION.

Year.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Period.	No. Prec. Sta.
1900-01.....	3.10	4.29	2.72	1.65	0.76	5.20	6.12	6.24	1.86	5.01	7.28	4.01	48.24	8
1901-02.....	2.74	2.40	6.76	2.16	3.89	4.31	3.79	3.06	5.41	7.25	3.45	6.27	51.43	7
1902-03.....	4.67	0.89	5.87	3.22	4.24	4.65	2.25	1.12	10.42	4.05	7.01	1.94	50.33	7
1903-04.....	6.49	2.46	3.79	3.35	2.65	3.29	3.84	3.26	5.02	3.72	5.19	6.42	49.48	7
1904-05.....	2.88	1.92	2.76	4.89	1.26	3.04	2.53	1.19	4.59	4.97	4.87	4.87	39.77	7
Mean.....	3.98	2.39	4.38	3.05	2.56	4.10	3.71	2.97	5.46	5.00	5.56	4.70	47.86	.....
Maximum....	6.49	4.29	6.76	4.89	4.24	5.20	6.12	6.24	10.42	7.25	7.28	6.42	51.49	.....
Minimum....	2.74	0.89	2.72	1.65	0.76	3.04	2.25	1.12	1.86	3.72	3.45	1.94	39.77	.....

RUN-OFF.

1900-01.....	(0.40)	0.96	1.76	0.98	0.49	4.54	6.43	4.50	2.11	0.76	1.51	1.18	25.68	.....
1901-02.....	1.76	1.20	4.13	3.21	(1.53)	8.77	3.99	2.46	1.48	2.19	1.37	1.08	33.17	.....
1902-03.....	2.36	1.56	3.94	2.96	(3.68)	7.00	3.82	1.11	4.28	2.49	2.19	1.55	36.94	.....
1903-04.....	3.25	1.92	2.54	(1.36)	(1.27)	(4.24)	4.20	2.50	2.42	1.36	1.05	1.50	27.61	.....
1904-05.....	1.68	1.33	(1.03)	(3.07)	(1.06)	(4.85)	4.73	1.41	0.92	0.56	0.87	2.25	23.76	.....
Mean.....	1.89	1.39	2.68	2.32	1.61	5.88	4.63	2.40	2.24	1.47	1.41	1.51	29.43	.....
Maximum....	3.25	1.92	4.13	3.21	(3.68)	8.77	6.43	4.50	4.28	2.49	2.19	2.25	36.94	.....
Minimum....	(0.40)	0.96	(1.03)	0.98	0.49	(4.24)	3.82	1.11	0.92	0.56	0.87	1.08	23.76	.....

Run-off, in Percentage of Precipitation.

1891-92	26	60	73	86	58	60	248	54	53	17	19	23	55	.....
1892-93	30	15	61	32	53	167	118	72	27	15	6	29	53	.....
1893-94	32	35	72	58	47	277	69	59	82	20	15	14	54	.....
1894-95	27	105	51	72	209	214	158	42	14	17	6	11	60	.....
1895-96	11	11	24	133	52	78	324	21	21	14	36	5	44	.....
1896-97	43	57	100	50	86	131	85	55	25	12	23	11	52	.....
1897-98	24	23	68	77	128	123	83	60	33	12	20	26	53	.....
1898-99	28	59	69	93	62	123	191	35	16	14	9	8	50	.....
1899-00	12	32	52	120	75	92	178	42	21	9	8	12	52	.....
1900-01	6	25	83	37	93	110	108	54	86	16	26	34	51	.....
1901-02	44	30	63	77	60	192	110	45	10	47	47	13	59	.....
1902-03	41	79	67	56	109	140	139	54	20	35	19	13	63	.....
1903-04	44	63	39	44	81	142	105	53	40	17	13	13	52	.....
1904-05	27	54	19	(63)	(41)	(175)	100	37	25	17	24	37	50	.....
Mean	30	41	59	71	74	134	124	52	31	20	18	20	55	.....
Maximum	44	105	100	133	209	277	324	72	86	47	47	73	63	.....
Minimum	6	11	19	32	41	60	25	21	10	9	6	5	44	.....

Loss, in Inches.

1891-92	2.60	1.05	1.11	0.61	0.72	1.65	−2.21	2.74	2.68	3.84	3.70	1.78	20.27	.....
1892-93	0.66	2.95	0.50	1.56	2.14	−1.79	−0.74	1.68	2.29	2.77	4.57	2.13	18.72	.....
1893-94	1.87	1.31	0.75	0.95	1.56	−2.14	1.37	3.16	0.50	1.94	1.87	4.81	17.95	.....
1894-95	3.40	−0.11	1.59	0.92	−1.21	−2.04	−1.44	1.64	2.97	2.21	3.68	1.93	18.54	.....
1895-96	1.30	2.24	2.76	−0.62	2.13	0.89	−2.84	2.28	3.45	4.41	1.22	3.82	21.04	.....
1896-97	2.23	1.24	0.00	0.88	0.32	−1.01	0.44	2.14	2.42	3.99	2.38	2.59	17.62	.....
1897-98	0.91	3.42	1.03	0.84	−0.50	−0.79	0.50	1.90	1.84	2.74	5.10	1.52	18.51	.....
1898-99	4.16	1.32	0.76	0.16	1.22	−0.90	−1.48	2.26	2.72	2.38	3.73	3.40	19.73	.....
1899-00	1.48	1.83	1.41	−0.46	0.92	0.28	−1.18	1.28	2.34	3.34	2.80	1.24	15.28	.....
1900-01	3.14	3.34	0.36	1.14	0.06	−0.37	−0.37	2.61	0.40	3.31	4.64	1.99	20.25	.....
1901-02	0.80	1.61	2.10	0.53	1.37	−3.58	−0.29	0.89	5.57	3.99	1.47	3.58	18.04	.....
1902-03	2.42	0.27	1.50	1.42	−0.33	−1.82	−1.08	0.58	5.14	2.96	5.25	0.53	16.84	.....
1903-04	2.77	0.75	1.47	1.84	0.42	−1.46	−0.17	1.81	2.01	4.09	3.44	2.80	19.77	.....
1904-05	1.98	0.42	1.73	(1.26)	(0.76)	(−2.75)	0.00	1.55	3.81	3.88	3.58	3.01	18.23	.....
Mean	2.12	1.55	1.22	0.79	0.69	−1.13	−0.67	1.89	2.73	3.28	3.39	2.43	18.29	.....
Maximum	4.16	3.42	2.76	1.84	2.14	1.65	1.37	3.16	5.57	4.41	5.25	4.81	21.04	.....
Minimum	0.66	−0.11	0.00	−0.62	−1.21	−3.58	−2.84	0.58	0.40	1.94	1.22	0.53	13.54	.....

TABLE 2—(*Continued*).

SUSQUEHANNA RIVER, ABOVE HARRISBURG, PA. DRAINAGE AREA, 28 030 SQ. MILES.

PRECIPITATION.

Year.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Period.	No. Prec. Sta.
1891-92.....	3.49	2.63	4.13	4.40	1.72	4.11	1.49	5.97	5.71	4.62	4.60	2.30	45.17	24
1892-93.....	0.95	3.45	1.28	2.30	4.55	2.68	4.06	6.05	3.15	3.26	4.84	3.00	39.57	24
1893-94.....	2.76	2.03	2.69	2.25	2.93	1.21	4.41	7.70	2.81	2.42	2.19	5.61	39.01	24
1894-95.....	4.64	2.04	3.28	3.32	1.11	1.78	2.50	2.84	3.47	2.66	3.93	2.17	33.74	24
1895-96.....	1.46	2.52	3.65	1.90	4.49	3.98	1.27	2.89	4.34	5.14	1.92	4.01	37.57	24
1896-97.....	3.88	2.89	1.04	1.77	2.33	3.22	3.03	4.72	3.24	4.53	3.11	2.90	36.66	24
1897-98.....	1.19	4.42	3.27	3.65	1.79	3.46	2.97	4.74	2.77	3.12	6.35	2.04	39.77	24
1898-99.....	5.74	3.23	2.43	2.29	3.22	3.94	1.63	3.48	3.25	2.76	4.08	3.70	39.75	24
1899-00.....	1.68	2.70	2.95	2.28	3.69	3.52	1.52	2.20	2.95	3.68	3.04	1.41	31.62	24
1900-01.....	3.35	4.43	2.12	1.81	0.93	3.52	4.46	5.68	2.96	3.96	6.24	3.01	42.47	24
1901-02.....	1.43	2.30	5.63	2.31	3.41	3.88	2.87	1.63	6.17	7.24	2.76	4.12	43.75	24
1902-03.....	4.13	1.24	4.56	3.23	3.71	4.58	2.76	1.27	6.44	4.52	6.48	1.95	44.87	24
1903-04.....	4.94	2.02	2.42	3.31	2.16	3.43	3.28	3.82	3.37	4.95	3.94	3.20	40.84	24
1904-05.....	2.71	0.92	2.13	3.41	1.29	3.65	2.34	2.45	5.06	4.66	4.68	3.19	36.49	22
Mean.....	3.02	2.63	2.97	2.73	2.67	3.35	2.76	3.96	3.98	4.11	4.16	3.04	39.38	.....
Maximum....	5.74	4.43	5.63	4.40	4.55	4.58	4.46	7.70	6.44	7.24	6.48	5.61	45.17	.....
Minimum	0.95	0.92	1.04	1.77	0.93	1.21	1.27	1.27	2.77	2.42	1.92	1.41	31.62	.....

RUN-OFF.

1891-92.....	0.89	1.58	3.02	3.79	1.00	2.46	3.70	3.23	3.03	0.78	0.90	0.52	24.90	... .
1892-93.....	0.29	0.50	0.78	0.74	2.41	4.47	4.80	4.37	0.86	0.49	0.27	0.87	20.85	.....
1893-94.....	0.89	0.72	1.94	1.30	1.37	3.35	3.04	4.54	2.31	0.48	0.32	0.80	21.06	.....
1894-95.....	1.24	2.15	1.69	2.40	2.32	3.82	3.94	1.20	0.50	0.45	0.25	0.24	20.20	.....
1895-96.....	0.16	0.28	0.89	2.52	2.36	3.09	4.11	0.61	0.89	0.73	0.70	0.19	16.53	.....
1896-97.....	1.65	1.65	1.04	0.89	2.01	4.23	2.59	2.58	0.82	0.54	0.73	0.31	19.04	.....
1897-98.....	0.28	1.00	2.24	2.81	2.29	4.25	2.47	2.84	0.93	0.38	1.25	0.52	21.26	.....
1898-99.....	1.58	1.91	1.67	2.13	2.00	4.84	3.11	1.22	0.53	0.38	0.35	0.30	20.02	.....
1899-00.....	0.20	0.87	1.54	2.74	2.77	3.24	2.70	0.92	0.61	0.34	0.24	0.17	16.34	.....
1900-01.....	0.21	1.09	1.76	0.67	0.87	3.89	4.83	3.07	2.56	0.65	1.60	1.02	22.22	.....
1901-02.....	0.63	0.69	3.53	1.78	2.04	7.46	3.16	0.74	0.60	3.25	1.29	0.54	25.71	.....
1902-03.....	1.71	0.97	3.06	1.81	4.04	6.40	3.84	0.69	1.30	1.56	1.23	1.42	28.03	.....
1903-04.....	2.17	1.27	0.95	1.47	1.74	4.89	3.45	2.01	1.36	0.86	0.50	0.40	21.07	.....
1904-05.....	0.73	0.50	0.40	(2.15)	(0.53)	(6.40)	2.34	0.90	1.25	0.78	1.10	1.18	18.26	.....
Mean.....	0.90	1.08	1.75	1.94	1.98	4.48	3.43	2.07	1.25	0.83	0.77	0.61	21.09	....
Maximum....	2.17	2.15	3.53	3.79	4.04	7.46	4.83	4.54	3.03	3.25	1.60	1.42	28.03	.....
Minimum	0.16	0.28	0.40	0.67	0.53	2.46	2.34	0.61	0.50	0.34	0.24	0.17	16.34	.....

RUN-OFF, IN PERCENTAGE OF PRECIPITATION.

1899-00.....	7	31	47	88	87	78	200	39	14	7	7	11	47	.....
1900-01.....	4	26	140	201	148	121	96	47	58	9	14	16	57	.....
1901-02.....	25	22	88	158	81	223	98	26	10	43	4	13	61	.....
1902-03.....	46	85	76	131	127	132	117	36	18	21	24	74	65	.....
1903-04.....	53	67	65	76	197	194	128	50	39	9	14	13	65	.....
1904-05.....	44	58	40	(81)	(47)	149	117	33	23	13	14	42	50	.....
Mean.....	35	40	77	112	111	148	117	41	24	20	15	25	58	.....
Maximum....	53	85	140	201	197	223	200	50	58	43	24	74	65	.....
Minimum....	4	22	40	76	(47)	78	96	26	10	7	4	11	47	.....

LOSS, IN INCHES.

1899-00.....	1.72	1.78	1.70	0.29	0.46	0.80	−1.50	1.20	2.54	3.85	2.53	1.25	16.62	.....
1900-01.....	3.46	3.46	−0.93	−1.72	−0.56	−0.72	0.21	2.87	1.32	3.67	5.10	2.46	18.61	.....
1901-02.....	1.27	2.08	0.07	−1.16	0.58	−4.33	0.05	1.61	5.80	4.45	2.76	3.76	17.04	.....
1902-03.....	2.07	0.17	0.97	−0.81	−0.78	−1.51	−0.39	0.71	5.21	3.47	4.98	0.43	14.52	.....
1903-04.....	2.82	0.74	0.85	0.83	−1.93	−2.99	−0.77	1.83	2.00	4.53	3.65	3.22	14.78	.....
1904-05.....	1.67	0.50	1.34	(0.62)	(0.65)	−1.60	−0.41	1.38	4.00	4.30	3.98	1.95	18.38	.....
Mean.....	2.17	1.45	0.77	0.32	−0.26	−1.73	−0.46	1.60	3.39	4.04	3.83	2.18	16.66	.....
Maximum....	3.45	3.46	1.70	0.83	(0.65)	0.80	0.21	2.87	5.80	4.53	5.10	3.76	18.61	.....
Minimum....	1.27	0.17	−0.93	−1.72	−1.93	−4.33	−1.50	0.71	1.32	3.47	2.53	0.43	14.52	.....

TABLE 2—(*Continued*).

SUSQUEHANNA RIVER, ABOVE WILKES-BARRE, PA. DRAINAGE AREA, 9 810 SQ. MILES.

PRECIPITATION.

Year.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Period.	No. Prec. Sta.
1899-00.....	1.85	2.58	3.19	2.43	3.46	3.59	1.50	1.97	2.94	4.13	2.73	1.40	31.77	27
1900-01.....	3.58	4.70	2.29	1.69	1.17	3.36	4.67	5.39	3.11	4.03	5.96	2.94	42.89	27
1901-02.....	1.69	2.68	5.58	2.00	3.03	3.51	2.54	2.17	5.87	7.86	2.88	4.39	44.13	27
1902-03.....	3.83	1.13	4.04	2.64	2.93	4.77	2.30	1.11	6.38	4.39	6.51	1.67	41.70	27
1903-04.....	6.04	2.21	2.44	3.40	1.99	3.17	2.79	3.69	3.27	4.96	4.26	3.69	41.91	27
1904-05.....	3.00	1.18	2.24	3.25	1.22	3.23	2.38	2.07	5.21	4.94	4.62	3.39	36.73	24
Mean.....	3.33	2.41	3.30	2.57	2.30	3.60	2.70	2.73	4.46	5.05	4.49	2.90	39.85	.....
Maximum....	6.04	4.70	5.58	3.40	3.46	4.77	4.67	5.39	6.38	7.86	6.51	4.32	44.13	.....
Minimum....	1.69	1.13	2.24	1.69	1.17	3.17	1.50	1.11	2.94	4.03	2.73	1.40	31.77	.....

RUN-OFF.

1899-00.....	0.13	0.80	1.49	2.14	3.00	2.79	3.00	0.77	0.40	0.28	0.20	0.15	15.15	.....
1900-01.....	0.13	1.24	3.22	3.41	1.73	4.06	4.46	2.52	1.79	0.36	0.86	0.48	24.28	.....
1901-02.....	0.42	0.60	4.91	3.16	2.45	7.84	2.49	0.56	0.57	3.41	0.12	0.56	27.09	.....
1902-03.....	1.76	0.96	3.07	3.45	3.71	6.28	2.69	0.40	1.17	0.92	1.53	1.24	27.18	.....
1903-04.....	3.22	1.47	1.59	2.57	3.92	6.16	3.56	1.86	1.27	0.43	0.61	0.47	27.13	.....
1904-05.....	1.33	0.68	0.90	(2.63)	(0.57)	4.88	2.79	0.69	1.21	0.64	0.64	1.44	18.35	.....
Mean.....	1.16	0.96	2.53	2.89	2.56	5.33	3.17	1.13	1.07	1.01	0.66	0.72	23.19	.....
Maximum....	3.22	1.47	4.91	3.45	3.92	7.84	4.46	2.52	1.79	3.41	1.53	1.44	27.18	.....
Minimum....	0.13	0.60	0.90	2.14	(0.57)	2.79	2.49	0.40	0.40	0.28	0.12	0.15	15.15	.....

RUN-OFF, IN PERCENTAGE OF PRECIPITATION.

1895-96.....	12	12	25	77	52	74	276	38	33	22	58	7	42	.....
1896-97	64	63	102	50	59	139	85	65	19	13	23	10	55	.....
1897-98.....	23	27	66	87	146	123	86	50	25	14	17	25	57	.....
1898-99.....	24	63	55	99	50	144	168	41	15	13	7	10	50	.....
1899-00.....	11	41	48	116	70	83	208	45	27	12	82	17	52	.....
1900-01.....	10	42	85	58	43	126	116	58	66	16	22	39	55	.....
1901-02.....	49	29	76	59	49	200	123	51	12	54	37	9	63	.....
1902-03.....	23	34	62	66	123	163	109	34	26	38	23	56	63	.....
1903-04.....	39	68	30	56	86	147	106	69	38	27	9	9	61	.....
1904-05.....	21	60	15	(74)	(53)	(164)	90	49	37	26	25	26	53	.....
Mean.....	29	42	56	75	72	137	121	52	29	27	21	18	56	.....
Maximum....	64	68	102	116	146	200	276	69	66	54	82	56	63	.....
Minimum....	10	12	15	50	43	74	85	34	12	12	7	7	42	.....

LOSS, IN INCHES.

1895-96.....	1.11	2.13	2.82	0.34	1.92	1.02	—2.54	1.27	3.09	4.47	0.96	4.39	20.89	.....
1896-97.....	1.54	1.02	—0.03	1.03	1.20	—1.46	0.47	1.55	2.58	4.58	2.54	3.08	18.05	.....
1897-98.....	0.90	3.58	1.20	0.46	—0.71	—1.21	0.43	2.11	2.52	2.50	4.56	0.93	17.27	.....
1898-99.....	4.71	1.00	1.26	0.04	1.74	—1.73	—1.25	2.17	3.06	2.41	3.91	3.14	20.46	.....
1899-00.....	1.66	1.63	2.06	—0.39	1.11	0.67	—1.44	1.21	2.14	3.21	2.97	0.87	15.70	.....
1900-01.....	3.34	2.59	0.30	0.77	0.72	—0.86	—0.76	2.26	1.25	3.19	5.18	1.95	19.93	.....
1901-02.....	0.46	2.05	1.34	1.01	1.62	—4.05	—0.74	0.94	5.05	3.47	1.72	3.34	16.21	.....
1902-03.....	2.46	0.94	1.56	1.06	—0.84	—2.79	—0.30	1.14	4.46	3.31	4.21	0.92	16.13	.....
1903-04.....	2.62	0.81	1.64	1.50	0.33	—2.33	—0.26	1.22	2.31	3.43	3.00	2.40	16.65	.....
1904-05.....	1.73	0.21	1.85	(0.88)	(0.49)	(—2.21)	0.26	1.29	2.73	3.30	3.65	2.11	16.29	.....
Mean.....	2.05	1.60	1.40	0.67	0.76	—1.50	—0.61	1.52	2.91	3.39	3.27	2.31	17.76	.....
Maximum....	4.71	3.58	2.82	1.50	1.92	1.02	0.47	2.26	5.05	4.58	5.18	4.39	20.89	.....
Minimum....	0.46	0.21	—0.03	—0.39	—0.84	—4.05	—2.54	0.94	1.25	2.41	0.96	0.87	15.70	.....

TABLE 2—(*Continued*).

SUSQUEHANNA RIVER, ABOVE WILLIAMSPORT, PA. DRAINAGE AREA, 5 640 SQ. MILES.

PRECIPITATION.

Year.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Period.	No. Prec. Sta.
1895-96.....	1.26	2.42	3.74	1.51	4.00	3.84	1.44	2.06	4.48	5.75	2.26	4.70	37.46	8
1896-97.....	4.22	2.75	1.25	2.04	2.95	3.77	3.21	4.47	3.18	5.28	3.30	3.37	39.79	8
1897-98.....	1.16	4.91	3.54	3.69	1.54	5.20	2.98	4.26	3.37	2.92	5.47	1.23	40.27	8
1898-99.....	6.22	2.68	2.81	2.49	3.46	3.89	1.85	3.70	3.60	2.77	4.18	3.50	41.15	8
1899-00.....	1.87	2.77	3.95	2.46	3.71	3.87	1.33	2.22	2.94	3.63	3.24	1.05	33.04	8
1900-01.....	3.71	4.43	2.05	1.83	1.28	3.42	4.69	5.41	3.69	3.79	6.62	3.19	44.11	8
1901-02.....	0.89	2.89	5.48	2.46	3.19	4.04	3.24	1.90	5.72	7.58	2.72	3.68	43.79	8
1902-03.....	3.18	1.43	4.12	3.09	3.68	4.41	3.23	1.74	6.08	5.30	5.44	2.08	43.73	8
1903-04.....	4.32	2.55	2.36	3.44	2.30	5.03	4.44	3.69	3.73	4.70	3.32	2.63	42.51	8
1904-05.....	2.20	0.54	2.18	3.33	1.05	3.46	2.50	2.52	4.36	4.45	4.87	2.84	34.30	5
Mean.....	2.90	2.74	3.15	2.63	2.72	4.09	2.89	3.20	4.11	4.62	4.14	2.83	40.02	.....
Maximum....	6.22	4.91	5.48	3.69	4.00	5.20	4.69	5.41	6.08	7.58	6.62	4.70	44.11	.....
Minimum	0.89	0.54	1.25	1.51	1.05	3.42	1.33	1.74	2.94	2.77	2.26	1.05	33.04	.....

RUN-OFF.

1895-96.....	0.15	0.29	0.92	1.17	2.08	2.82	3.98	0.79	1.48	1.28	1.30	0.31	16.57	.....
1896-97.....	2.68	1.73	1.28	1.01	1.75	5.23	2.74	2.92	0.60	0.70	0.76	0 34	21.74	.....
1897-98.....	0.26	1.33	2.34	3.23	2.25	6.41	2.55	2.15	0.85	0.42	0.91	0.30	23.00	.....
1898-99.....	1.51	1.68	1.55	2.45	1.72	5.62	3.10	1.53	0.54	0.36	0.27	0.36	20.69	.....
1899-00.....	0.21	1.14	1.89	2.85	2.60	3.20	2.77	1.01	0.80	0.42	0.27	0.18	17.34	.....
1900-01.....	0.37	1.84	1.75	1.06	0.56	4.38	5.45	3.15	2.44	0.60	1.44	1.24	24.18	.....
1901-02.....	0.43	0.84	4.14	1.45	1.57	8.09	3.98	0.96	0.67	4.11	1.00	0.34	27.58	.....
1902-03.....	0.72	0.49	2.56	2.03	4.52	7.20	3.53	0.60	1.57	1.99	1.23	1.16	27.60	.....
1903-04.....	1.70	1.74	0.72	1.94	1.97	7.38	4.70	2.47	1.42	1.27	0.32	0.23	25.86	.....
1904-05.....	0.47	0.33	0.33	(2.45)	(0.56)	(5.67)	2.24	1.23	1.63	1.15	1.22	0.73	18.01	.....
Mean.....	0.85	1.14	1.75	1.96	1.96	5.59	3.50	1.68	1.20	1.23	0.87	0.52	22.26	.....
Maximum....	2.68	1.84	4.14	3.23	4.52	8.09	5.45	3.15	2.44	4.11	1.44	1.24	27.60	.....
Minimum	0.15	0.29	0.33	1.01	0.56	2.82	2.24	0.60	0.54	0.36	0.27	0.18	16.57	.....

RUN-OFF.

1884-85.....	0.39	0.40	1.64	3.38	1.07	1.37	4.34	1.31	1.24	0.44	1.44	0.80	17.82	.....
1885-86.....	0.97	1.43	2.20	2.93	2.98	2.40	4.60	1.38	0.87	0.51	0.62	0.29	21.18	.....
1886-87.....	0.31	2.16	2.09	2.72	6.71	2.87	1.80	1.88	1.14	0.28	0.26	0.17	22.39	.....
1887-88.....	(0.16)	0.27	0.53	2.76	2.40	2.55	2.71	1.27	0.64	1.61	1.54	1.02	17.46	.....
1888-89.....	2.11	2.67	2.03	2.55	1.91	2.87	2.76	1.72	3.37	1.41	0.66	0.28	24.34	.....
1889-90.....	0.53	2.90	3.30	4.30	4.40	4.98	3.53	5.10	1.45	0.48	0.73	2.50	34.20	.....
1890-91.....	3.70	2.97	1.48	3.92	7.29	3.29	2.97	0.51	1.80	1.21	0.98	0.52	30.64	.....
1891-92.....	0.22	0.92	2.55	2.63	2.78	2.95	2.59	3.46	2.66	0.74	0.35	0.24	22.09	.....
1892-93.....	0.14	0.57	(1.24)	(1.28)	(5.42)	3.78	3.24	4.36	0.65	0.39	0.16	0.29	21.52	.....
1893-94.....	0.55	0.57	2.36	1.94	2.64	2.69	2.36	2.62	1.17	0.28	0.21	0.64	18.03	.....
1894-95.....	0.30	0.76	1.57	3.88	0.78	3.79	3.21	0.71	0.31	0.45	0.28	0.25	16.29	.....
1895-96.....	0.15	0.43	1.44	1.48	2.84	3.04	3.47	0.94	0.97	3.01	1.59	0.35	19.71	.....
1896-97.....	1.68	1.29	1.74	1.43	4.05	4.77	2.62	2.17	0.75	1.11	0.77	0.21	22.59	.....
1897-98.....	0.12	0.91	2.56	4.85	2.88	5.02	2.50	2.36	0.84	0.38	1.88	0.32	24.61	.....
1898-99.....	1.15	2.09	2.31	3.42	2.53	4.66	2.56	1.79	1.00	0.66	0.40	0.31	22.88	.....
1899-00.....	0.20	0.77	2.17	(3.04)	2.95	3.84	1.94	0.65	0.79	0.78	0.36	0.15	17.64	.....
1900-01.....	0.16	1.65	1.88	1.80	0.83	4.58	6.81	2.63	2.16	0.61	0.58	0.86	24.55	.....
1901-02.....	0.33	0.85	3.64	1.45	1.64	6.31	4.03	1.08	0.72	3.49	0.91	0.21	24.66	.....
1902-03.....	0.75	0.51	3.56	2.70	4.95	6.26	3.32	0.67	1.33	1.56	0.65	0.83	27.09	.....
1903-04.....	0.74	1.03	0.83	4.32	2.10	6.54	3.64	2.47	1.62	1.44	0.37	0.25	25.35	.....
1904-05.....	0.38	0.24	0.78	1.63	(2.31)	6.89	2.20	1.63	1.80	1.33	1.26	0.74	21.19	.....
Mean.....	0.72	1.21	1.99	2.78	3.12	4.07	3.20	1.94	1.30	1.06	0.76	0.53	22.68	.....
Maximum....	3.70	2.97	3.64	4.30	7.29	6.89	6.81	5.10	3.37	3.49	1.88	2.50	34.20	.....
Minimum	0.12	0.24	0.53	(1.28)	0.78	1.37	1.80	0.51	0.31	0.28	0.16	0.15	16.29	.....

TABLE 2—(*Continued*).

OHIO RIVER, ABOVE WHEELING, W. VA. DRAINAGE AREA, 23 820 SQ. MILES.

PRECIPITATION.

Year.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Period.	No. Prec. Sta.
1884–85.....	2.53	1.76	3.67	4.27	1.97	1.40	2.78	3.81	4 19	3.59	6.88	2.27	39.12	15
1885–86.....	4.93	2.82	2.32	3.93	1.98	2.88	3.19	3.33	3.85	3.88	3.11	3.67	39.89	25
1886–87.....	1.21	4.91	2.98	2.14	5.54	1.94	3.11	2.94	4.41	3.18	3.14	2.84	39.34	25
1887–88.....	1.72	2.15	1.84	4.67	1.76	2.81	2.38	3.86	3.21	4.39	4.98	3.69	37.46	26
1888–89.....	3.84	3.85	2.06	3.09	1.94	2.17	3.64	4.03	5.01	4.27	2.31	3.81	39.52	27
1889–90.....	2.62	4.56	3.53	4.62	5.01	4.66	4.21	7.48	3.91	2.92	5.56	6.48	55.56	37
1890–91.....	6.78	2.60	3.73	3.04	6.01	3.70	2.22	2.81	5.15	6.23	3.91	2.45	48.63	40
1891–92.....	2.23	3.49	3.88	3.26	2.78	3.06	3.01	6.54	5.43	4.18	3.09	2.45	43.40	36
1892–93.....	1.36	3.12	2.07	2.76	5.83	1.97	5.34	5.31	3.36	3.08	3.44	1.91	39.40	42
1893–94.....	3.91	2.09	3.38	2.87	3.19	2.63	3.68	5.76	2.50	2.54	1.30	4.70	38.55	47
1894–95.....	2.53	2.54	3.82	4.22	1.19	2.34	2.68	2.18	3.12	3.38	3.41	2.06	33.47	51
1895–96.....	1.13	3.55	3.57	1.72	3.21	3.73	2.70	2.97	5.23	9.08	2.83	4.85	44.57	48
1896–97.....	2.35	3.01	1.95	2.16	3.66	3.85	3.31	3.61	3.43	6.29	3.15	1.67	38.44	52
1897–98	0.51	5.67	3.74	4.97	2.28	5.62	2.73	4.28	4.09	3.68	6.69	2.25	46.51	57
1898–99.....	4.66	3.31	2.73	3.19	2.97	4.61	1.89	4.78	4.03	4.60	2.38	4.11	43.26	54
1899–00.....	1.70	2.42	3.76	2.58	3.78	3.55	1.57	2.45	4.10	5.31	3.16	1.56	35.94	54
1900–01.....	2.78	5.09	1.98	2.45	1.20	3.69	6.50	5.25	4.51	3.15	4.91	3.75	45.21	57
1901–02.....	0.72	3.00	4.64	2.55	2.14	3.51	3.55	3.05	5.63	6.63	2.90	3.06	40.78	58
1902–03.....	3.16	1.99	5.07	3.02	4.81	4.38	8.45	2.59	5.80	4.94	4.35	1.64	45.20	60
1903–04.....	8.04	2.91	2.22	3.51	2.93	4.98	3.87	3.95	4.08	4.58	3.33	2.27	41.62	65
1904–05.....	2.15	0.65	2.84	3.02	1.87	3.70	3.07	3.91	5.72	5.77	4.26	2.89	39.91	62
Mean.....	2.66	3.09	3.13	3.24	3.19	3.39	3.28	4.04	4.32	4.55	3.74	3.07	41.71	.....
Maximum....	6.78	5.67	5.07	4.97	6.54	5.62	6.50	7.48	5.80	9.08	6.88	6.48	55.56	.....
Minimum	0.51	0.65	1.84	1.72	1.19	1.40	1.57	2.18	2.50	2.54	1.30	1.56	33.47	.....

Loss, in Inches.

1884–85.....	2.14	1.36	2.03	0.89	0.90	0.03	−1.56	2.50	2.95	3.15	5.44	1.47	21.30	.....
1885–86.....	3.96	1.39	0.12	1.00	−1.00	0.48	−1.41	1.95	2.98	3.37	2.49	3.38	18.71	.....
1886–87..	0.90	2.75	0.89	−0.58	−0.17	−0.93	1.31	1.06	3.27	2.90	2.88	2.67	16.95	.....
1887–88.....	(1.56)	1.88	1.31	1.91	−0.64	0.26	−0.33	2.59	2.57	2.78	3.44	2.67	20.00	.....
1888–89.....	1.73	0.68	0.03	0.54	0.03	−0.70	0.88	2.31	1.64	2.86	1.65	3.53	15.18	.....
1889–90.....	2.09	1.66	0.23	0.32	0.61	−0.32	0.68	2.38	2.46	2.44	4.83	3.98	21.36	.....
1890–91.....	3.08	−0.37	2.25	−0.88	−1.28	0.41	−0.75	2.30	3.35	5.02	2.93	1.93	17.99	.....
1891–92.....	2.01	2.57	1.33	0.63	0.00	0.11	0.42	3.08	2.77	3.44	2.74	2.21	21.31	.....
1892–93.....	1.22	2.55	(0.83)	(1.48)	(0.41)	−1.81	2.00	0.95	2.71	2.64	3.28	1.62	17.88	.....
1893–94.....	3.36	1.52	1.02	0.93	0.55	−0.06	1.32	3.14	1.33	2.26	1.09	4.06	20.52	.....
1894–95.....	2.23	1.78	2.25	0.34	0.41	−1.45	−0.53	1.47	2.81	2.93	3.13	1.81	17.18	.....
1895–96.....	0.98	3.12	2.13	0.24	0.37	0.69	−0.77	2.03	4.26	6.07	1.24	4.50	24.86	.....
1896–97.....	0.67	1.72	0.21	0.73	−0.39	−0.22	0.69	1.44	2.68	5.18	2.38	1.46	15.85	.....
1897–98.....	0.39	4.76	1.19	0.12	−0.60	0.60	0.23	1.92	3.25	3.30	4.81	1.93	21.90	.....
1898–99.....	3.51	1.22	0.42	(−0.23)	0.44	−0.05	−0.67	2.99	3.03	3.94	1.98	3.80	20.38	.....
1899–00.....	1.50	1.65	1.59	−0.46	0.83	−0.29	−0.37	1.80	3.31	4.53	2.80	1.41	18.30	.....
1900–01.....	2.62	3.44	0.05	0.65	0.37	−0.89	−0.31	2.62	2.35	2.54	4.33	2.89	20.48	.....
1901–02.....	0.39	2.15	1.00	1.10	0.50	−2.80	−0.48	1.97	4.91	3.14	1.39	2.85	16.12	.....
1902–03.....	2.41	1.48	1.51	0.32	−0.14	−1.88	0.13	1.92	4.47	3.38	3.70	0.81	18.11	.....
1903–04.....	2.30	1.88	1.39	−0.81	0.83	−1.61	0.23	1.48	2.46	3.14	2.96	2.02	16.27	.....
1904–05.....	1.77	0.41	2.06	1.39	(−0.44)	−3.13	0.87	2.28	3.92	4.44	3.00	2.15	18.72	.....
Mean.....	1.94	1.88	1.14	0.46	0.07	−0.68	0.07	2.10	3.02	3.50	2.98	2.53	19.02	.....
Maximum....	3.96	4.76	2.25	1.91	0.90	0.69	2.00	3.14	4.91	6.07	5.44	4.50	24.86	.....
Minimum....	0.39	−0.37	0.03	−0.88	−1.28	−3.13	−1.56	0.95	1.33	2.26	1.09	0.81	15.18	.....

TABLE 2—(*Continued*).

OHIO RIVER, ABOVE WHEELING, W. VA. DRAINAGE AREA, 23 820 SQ. MILES.

RUN-OFF, IN PERCENTAGE OF PRECIPITATION.

Year.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Period.	No. Prec. Sta.
1884-85.....	15	23	45	79	54	98	156	34	30	12	21	35	46	.....
1885-86.....	20	51	95	75	151	83	144	41	23	13	20	8	53	.....
1886-87.....	26	44	70	127	103	148	58	64	26	9	8	6	57	.....
1887-88.....	(9)	13	29	59	136	91	114	33	20	37	31	28	47	.....
1888-89.....	55	80	99	83	98	132	76	43	67	33	29	7	62	.....
1889-90.....	20	64	93	93	88	107	84	68	37	16	13	39	62	.....
1890-91.....	55	114	40	129	121	89	134	18	35	19	25	21	63	.....
1891-92.....	10	26	66	81	100	96	86	53	49	18	11	10	51	....
1892-93.....	10	18	(60)	(46)	(93)	191	62	82	19	13	5	15	55	.....
1893-94.....	14	27	70	68	83	102	64	45	47	11	16	14	47	.....
1894-95.....	12	30	41	92	66	116	120	33	10	13	7	12	49	.....
1895-96.....	13	12	40	86	88	82	129	32	19	33	56	7	44	.....
1896-97.....	72	43	89	66	111	124	79	60	22	18	24	13	59	.....
1897-98.....	24	16	68	98	126	89	92	55	21	10	28	14	53	.....
1898-99.....	25	63	85	(107)	85	101	135	57	25	14	17	8	53	.....
1899-00.....	12	32	58	118	78	108	124	27	19	15	11	10	49	....
1900-01.....	6	32	97	73	69	124	105	50	48	19	12	23	54	.....
1901-02.....	46	28	78	57	77	180	114	35	13	53	40	7	60	.....
1902-03.....	24	26	70	89	103	143	96	26	23	32	15	51	60	.....
1903-04.....	24	35	37	123	72	133	94	63	40	31	11	11	61	.....
1904-05..	18	37	27	54	(124)	183	72	42	31	23	30	26	53	.....
Mean.....	27	39	64	86	98	120	98	48	30	23	20	17	54	.....
Maximum....	72	114	99	129	151	191	156	82	67	53	56	39	63	.....
Minimum	6	12	27	(46)	54	82	58	18	10	9	5	6	44	.....

Run-off, in Percentage of Precipitation.

1895-96.....	12	11	10	32	33	(49)	(84)	(11)	13	19	24	4	22	.....
1896-97.....	162	26	76	33	78	104	55	57	28	16	17	17	51	.....
1897-98.....	17	6	24	46	84	41	78	45	27	9	38	22	35	.....
1898-99.....	25	41	63	98	70	106	99	27	18	13	10	7	44	.....
1899-00.....	11	22	25	47	42	64	80	23	23	10	11	8	30	.....
1900-01.....	8	15	42	30	85	48	76	50	48	34	16	28	39	.....
1901-02.....	90	22	54	75	90	149	104	37	11	13	13	8	61	.....
1902-03.....	10	13	50	58	69	89	89	25	32	33	16	24	43	.....
1903-04.....	15	27	45	42	163	64	35	33	24	12	12	8	32	.....
1904-05.....	8	19	10	33	35	97	44	18	15	18	17	20	26	.....
Mean.....	24	19	40	51	69	82	76	36	24	18	20	13	39	.....
Maximum....	162	41	76	98	163	149	104	57	48	34	38	28	61	.....
Minimum	8	6	10	30	33	41	35	(11)	11	9	10	4	22	.....

Loss, in Inches.

1895-96.....	1.05	1.22	2.32	1.34	2.32	(1.81)	(0.25)	(2.60)	4.38	4.68	1.28	5.84	29.09	.....
1896-97.....	−0.57	2.22	0.18	1.04	1.28	−0.09	1.03	2.03	1.74	3.44	2.44	1.33	16.07	.....
1897-98.....	1.19	3.86	2.44	2.03	0.17	2.60	0.52	2.60	1.33	2.92	4.34	1.03	25.03	.....
1898-99.....	4.78	1.42	1.08	0.05	1.41	−0.25	0.01	3.77	2.86	1.98	2.56	3.67	23.32	.....
1899-00.....	1.68	1.32	1.46	1.12	2.00	1.22	0.27	1.79	3.22	3.38	1.85	1.79	21.10	.....
1900-01.....	1.92	3.09	1.03	1.36	0.07	1.81	1.45	3.25	2.38	2.54	5.23	2.27	26.40	.....
1901-02.....	0.06	1.96	2.65	0.69	0.37	−2.15	−0.14	1.26	3.08	2.38	1.87	1.84	13.87	.....
1902-03.....	3.03	2.16	2.24	1.46	1.09	0.40	0.41	2.18	4.49	3.15	3.09	1.75	25.45	.....
1903-04.....	2.19	0.66	0.43	1.18	−0.75	0.74	1.60	2.31	3.61	4.00	2.03	2.07	20.07	.....
1904-05.....	1.52	0.64	2.26	2.09	1.12	0.09	0.96	2.51	4.47	5.41	3.38	1.49	25.94	.....
Mean.....	1.68	1.85	1.61	1.23	0.91	0.62	0.63	2.43	3.16	3.39	2.81	2.31	22.64	.....
Maximum....	4.78	3.86	2.65	2.09	2.32	2.60	1.60	3.77	4.49	5.41	5.23	5.84	29.09	.....
Minimum	−0.57	0.64	0.18	0.05	−0.75	−2.15	−0.14	1.26	1.33	1.98	1.27	1.03	13.87	.....

TABLE 2—(*Continued*).

POTOMAC RIVER, ABOVE POINT OF ROCKS, MD. DRAINAGE AREA, 9 650 SQ. MILES.

PRECIPITATION.

Year.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Period.	No. Prec. Sta.
1895–96.....	1.19	1.37	2.59	1.97	3.48	3.55	1.61	2.91	5.01	5.79	1.69	6.09	37.25	10
1896–97.....	0.92	3.02	0.74	1.55	5.88	2.40	2.28	4.77	2.43	4.08	2.93	1.60	32.60	12
1897–98.....	1.43	4.10	3.22	3.78	1.07	4.44	2.36	4.76	1.81	3.21	7.00	1.32	38.50	12
1898–99.....	6.41	2.41	2.91	2.54	4.44	3.96	1.37	5.15	3.47	2.28	2.84	3.94	41.72	13
1899–00.....	1.88	1.69	1.94	2.10	3.44	3.42	1.34	2.32	4.19	3.74	2.08	1.95	30.09	15
1900–01.....	2.08	3.62	1.77	1.95	0.46	3.46	6.05	6.47	4.56	3.82	6.23	3.15	43.63	15
1901–02.....	0.57	2.50	5.71	2.78	3.88	4.35	3.20	1.97	3.45	2.75	2.16	2.01	35.33	15
1902–03.....	3.36	2.49	4.50	3.51	3.49	3.59	3.75	2.92	6.57	4.67	3.67	2.29	44.81	14
1903–04.....	2.57	0.91	0.78	2.05	1.20	2.08	2.46	3.43	4.78	4.54	2.32	2.25	29.37	14
1904–05.....	1.66	0.79	2.52	3.12	1.73	2.89	1.72	3.05	5.23	6.63	4.08	1.86	35.28	13
Mean.....	2.21	2.29	2.67	2.53	2.91	3.4	2.61	3.77	4.15	4.15	3.50	2.65	36.86	.....
Maximum....	6.41	4.10	5.71	3.78	5.88	4.44	6.05	6.47	6.57	6.63	7.00	6.09	44.81	.....
Minimum	0.57	0.79	0.74	1.55	0.46	2.08	1.34	1.97	1.81	2.28	1.69	1.32	29.37	.....

RUN-OFF.

1895–96.....	0.14	0.15	0.27	0.63	1.16	(1.74)	(1.36)	(0.31)	0.68	1.11	0.41	0.25	8.16	.....
1896–97.....	1.49	0.80	0.56	0.51	4.60	2.49	1.25	2.74	0.69	0.64	0.49	0.27	16.53	.....
1897–98.....	0.24	0.24	0.78	1.75	0.90	1.84	1.84	2.16	0.48	0.29	2.66	0.29	13.47	.....
1898–99.....	1.63	0.99	1.83	2.49	3.03	4.21	1.36	1.38	0.61	0.30	0.28	0.27	18.38	.....
1899–00.....	0.20	0.37	0.48	0.98	1.44	2.20	1.07	0.53	0.97	0.36	0.23	0.16	8.99	.....
1900–01.....	0.16	0.53	0.74	0.59	0.39	1.65	4.60	3.22	2.18	1.28	1.00	0.88	17.22	.....
1901–02.....	0.51	0.54	3.06	2.09	3.51	6.50	3.34	0.71	0.37	0.37	0.29	0.17	21.46	.....
1902–03.....	0.33	0.33	2.26	2.05	2.40	3.19	3.34	0.74	2.08	1.52	0.58	0.54	19.36	.....
1903–04.....	0.38	0.25	0.35	0.87	1.95	1.34	0.86	1.12	1.17	0.54	0.29	0.18	9.30	.....
1904–05.....	0.14	0.15	0.26	1.03	0.61	2.80	0.76	0.54	0.76	1.22	0.70	0.37	9.34	.....
Mean.....	0.52	0.44	1.06	1.30	2.00	2.80	1.98	1.34	0.99	0.76	0.69	0.34	14.22	.....
Maximum....	1.63	0.99	3.06	2.49	4.60	6.50	4.60	3.22	2.18	1.52	2.66	0.88	21.46	.....
Minimum	0.14	0.15	0.26	0.51	0.39	1.34	0.76	(0.31)	0.37	0.29	0.23	0.16	8.16	.....

Run-off, in Percentage of Precipitation.

1895–96.....	20	15	9	38	35	35	85	36	13	18	14	5	21	.....
1896–97.....	623	16	383	33	(62)	81	53	47	19	13	27	25	51	.....
1897–98.....	10	12	18	31	75	23	61	42	21	11	41	26	30	.....
1898–99.....	31	59	55	78	(26)	(53)	106	20	29	14	10	10	34	.....
1899–00.....	13	64	40	44	36	61	55	27	14	14	17	8	29	.....
1900–01.....	11	18	47	30	139	34	77	58	41	39	33	25	40	.....
1901–02.....	99	25	51	74	67	148	113	38	18	19	18	10	58	.....
1902–03.....	11	13	54	64	54	73	81	33	36	43	17	35	44	.....
1903–04.....	23	44	38	(27)	(71)	(49)	31	31	16	15	17	12	23	.....
1904–05.....	17	21	12	28	27	69	38	14	17	19	19	18	23	.....
Mean.....	33	23	40	47	49	61	69	36	24	20	23	15	36	.....
Maximum....	623	64	383	78	139	148	113	65	41	43	41	41	58	.....
Minimum....	10	12	9	(27)	(26)	23	31	14	13	11	10	5	21	.....

Loss, in Inches.

1895–96.....	0.85	1.34	2.80	1.55	2.44	2.77	0.19	1.46	4.95	5.11	2.76	6.83	33.05	.....
1896–97.....	—2.51	3.48	—0.65	0.94	(2.36)	0.44	0.90	2.73	2.57	3.59	1.03	0.76	15.68	.....
1897–98.....	2.32	2.27	2.89	2.11	0.19	2.87	1.04	3.01	2.25	3.70	4.58	1.46	28.64	.....
1898–99.....	4.91	0.74	1.40	0.55	(3.67)	(2.40)	—0.07	3.97	1.49	2.17	3.56	3.41	28.20	.....
1899–00.....	2.04	0.29	0.80	1.43	2.40	1.46	0.85	1.77	5.14	3.23	1.61	2.88	23.90	.....
1900–01.....	2.83	2.40	0.97	1.61	—0.13	2.50	1.45	2.46	4.47	2.67	4.56	2.85	28.64	.....
1901–02.....	0.01	1.51	3.00	0.70	1.46	—1.73	—0.30	1.64	2.41	1.75	1.81	2.32	14.58	.....
1902–03.....	3.24	2.78	1.56	1.46	1.59	1.11	0.69	1.80	4.85	1.73	2.92	1.58	25.32	.....
1903–04.....	1.88	0.46	0.60	(1.32)	(0.36)	(1.06)	1.82	2.36	4.67	4.30	2.11	1.72	22.61	.....
1904–05.....	1.00	0.75	2.16	2.06	1.56	0.76	1.18	3.40	4.55	4.83	2.59	1.35	26.19	.....
Mean.....	1.65	1.60	1.56	1.37	1.59	1.36	0.78	2.46	3.74	3.31	2.75	2.52	24.69	.....
Maximum....	4.91	3.48	3.00	2.11	(3.67)	(2.40)	1.82	3.97	5.14	5.11	4.58	6.83	33.05	.....
Minimum....	—2.51	0.29	—0.65	0.55	—0.13	—1.73	—0.30	1.46	1.49	1.73	1.03	0.76	14.58	.....

TABLE 2—(*Continued*).

SHENANDOAH RIVER, ABOVE MILLVILLE, W. VA. DRAINAGE AREA, 3 000 SQ. MILES.

PRECIPITATION.

Year.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Period.	No. Prec. Sta.
1895-96.....	1.06	1.57	3.09	2.49	3.78	4.25	1.24	2.28	5.71	6.21	3.30	7.22	42.10	4
1896-97.....	0.48	4.16	0.23	1.41	6.28	2.34	1.90	5.13	3.18	4.14	1.41	1.01	31.67	4
1897-98.....	2.59	2.59	3.51	3.05	0.75	3.73	2.66	5.21	2.86	4.18	7.73	1.97	40.83	5
1898-99.....	7.10	1.79	3.13	2.52	4.95	5.12	1.16	4.95	2.09	2.51	3.94	3.78	43.04	5
1899-00.....	2.34	0.81	1.34	2.57	3.73	3.72	1.89	2.42	5.98	3.76	1.93	3.14	33.63	5
1900-01.....	3.19	2.94	1.83	2.31	0.33	3.80	6.24	5.82	7.54	4.38	5.92	3.78	48.08	5
1901-02.....	0.69	2.01	6.12	2.69	4.46	3.61	2.23	2.64	2.93	2.17	2.22	2.59	34.36	4
1902-03.....	3.65	3.21	3.38	4.08	3.49	4.15	3.62	2.69	7.63	3.06	3.53	2.42	44.91	6
1903-04.....	2.39	0.82	0.96	1.80	1.26	2.08	2.64	3.43	5.57	5.04	2.53	1.95	30.47	7
1904-05.....	1.20	0.95	2.46	2.86	2.14	2.44	1.90	3.93	5.50	5.98	3.19	1.65	34.20	7
Mean.....	2.47	2.08	2.61	2.58	3.12	3.52	2.55	3.85	4.90	4.14	3.56	2.95	38.33	.....
Maximum....	7.10	4.16	6.12	4.08	6.28	5.12	6.24	5.82	7.63	6.21	7.73	7.22	48.08	.....
Minimum	0.48	0.81	0.23	1.41	0.33	2.08	1.16	2.28	2.09	2.17	1.41	1.01	30.47	.....

RUN-OFF.

1895-96.....	0.21	0.23	0.29	0.94	1.34	1.48	1.05	0.82	0.76	1.10	0.44	0.39	9.05	.....
1896-97.....	2.99	0.68	0.88	0.47	(3.92)	1.90	1.00	2.40	0.61	0.55	0.38	0.25	16.03	.....
1897-98.....	0.27	0.32	0.62	0.94	0.56	0.86	1.62	2.20	0.61	0.48	3.15	0.51	12.14	.....
1898-99.....	2.19	1.05	1.73	1.97	(1.28)	(2.72)	1.23	0.98	0.60	0.34	0.38	0.37	14.84	.....
1899-00.....	0.30	0.52	0.54	1.14	1.33	2.26	1.04	0.65	0.84	0.53	0.32	0.26	9.73	.....
1900-01.....	0.36	0.54	0.86	0.70	0.46	1.30	4.79	3.36	3.07	1.71	1.36	0.93	19.44	.....
1901-02.....	0.68	0.50	3.12	1.99	3.00	5.34	2.53	1.00	0.52	0.42	0.41	0.27	19.78	.....
1902-03.....	0.41	0.43	1.82	2.62	1.90	3.04	2.93	0.89	2.78	1.33	0.61	0.84	19.60	.....
1903-04.....	0.56	0.36	0.36	(0.48)	(0.90)	(1.02)	0.82	1.07	0.90	0.74	0.42	0.23	7.86	.....
1904-05.....	0.20	0.20	0.30	0.80	0.58	1.68	0.72	0.53	0.95	1.15	0.60	0.30	8.01	.....
Mean.....	0.82	0.48	1.05	1.21	1.53	2.16	1.77	1.39	1.16	0.83	0.81	0.43	13.64	.....
Maximum....	2.99	1.05	3.12	2.62	3.63	5.34	4.79	3.36	3.07	1.71	3.15	0.93	19.78	.....
Minimum ...	0.20	0.20	0.29	0.47	0.46	0.86	0.72	0.53	0.52	0.34	0.38	0.23	7.86	.....

RUN-OFF, IN PERCENTAGE OF PRECIPITATION.

1898-99.....	(28)	(58)	(47)	69	76	83	101	37	19	13	11	14	44	.....
1899-00.....	13	69	29	49	48	73	67	32	27	17	19	11	37	.....
1900-01.....	19	41	55	61	108	44	64	51	54	28	30	33	44	.....
1901-02.....	131	37	46	74	60	128	95	40	21	17	16	8	50	.....
1902-03.....	22	25	70	62	64	81	88	66	31	37	23	44	49	.....
1903-04.....	23	47	33	31	62	72	50	49	27	12	23	17	35	.....
1904-05.....	39	23	21	34	34	94	48	33	18	33	24	26	33	.....
Mean.....	25	36	46	56	60	81	71	43	29	24	23	21	42	.....
Maximum....	131	69	70	74	108	128	101	66	54	37	30	44	50	.....
Minimum ...	13	23	21	31	34	44	48	32	18	12	11	8	33	.....

LOSS, IN INCHES.

1898-99.....	(6.01)	(0.88)	(2.02)	1.11	1.18	1.09	—0.02	2.05	2.87	2.53	4.75	3.54	28.08	.....
1899-00.....	2.26	0.29	1.34	1.70	2.31	1.02	0.95	1.98	3.89	2.46	1.24	3.53	22.97	.....
1900-01.....	3.04	1.85	1.24	1.04	—0.05	1.96	2.47	3.25	2.57	3.78	7.14	2.50	30.79	.....
1901-02.....	—0.17	0.90	3.90	0.88	1.98	—0.92	0.10	1.55	3.04	1.94	2.34	3.46	18.90	.....
1902-03.....	4.69	2.67	1.01	1.67	1.81	0.99	0.40	0.60	5.37	1.94	3.38	1.33	35.86	.....
1903-04.....	1.83	0.49	0.95	1.53	0.66	0.73	1.24	1.80	3.29	3.87	1.85	1.65	19.89	.....
1904-05.....	0.33	0.89	1.91	1.98	2.05	0.14	1.08	3.59	4.34	4.98	3.66	2.03	26.98	.....
Mean.....	2.57	1.14	1.77	1.40	1.41	0.72	0.89	2.12	3.63	3.07	3.48	2.57	24.77	.....
Maximum....	(6.01)	2.67	3.90	1.93	2.31	1.96	2.47	3.59	5.37	4.98	7.14	3.54	30.79	.....
Minimum ...	—0.17	0.29	0.95	0.83	—0.05	—0.92	—0.02	0.60	2.57	1.94	1.24	1.33	18.90	.....

TABLE 2—(*Continued*).

JAMES RIVER, ABOVE CARTERSVILLE, VA. DRAINAGE AREA, 6 230 SQ. MILES.

PRECIPITATION.

Year.	Oct.	Nov.	Dec	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Period.	No. Prec. Sta.
1898-99.....	8.35	2.09	3.82	3.61	4.92	6.38	1.72	3.40	3.55	2.93	5.35	4.11	50.23	6
1899-00.....	2.62	0.93	1.90	3.35	4.41	3.76	2.84	2.92	5.34	2.98	1.54	3.98	36.57	10
1900-01.....	3.75	3.12	2.78	2.67	0.59	3.48	6.92	6.70	5.62	5.23	10.22	3.75	54.83	10
1901-02.....	0.55	1.43	7.19	3.21	4.79	3.30	2.05	2.58	3.88	2.35	2.80	3.75	37.83	10
1902-03.....	6.05	3.57	3.41	4.43	5.08	5.29	3.42	1.78	7.73	3.06	4.38	2.39	50.59	8
1903-04.....	2.38	0.93	1.41	2.21	1.76	2.59	2.46	3.53	4.53	4.41	2.39	1.98	30.58	8
1904-05.....	0.54	1.15	2.41	2.94	3.11	2.34	2.09	5.33	5.29	7.47	4.79	2.75	40.21	8
Mean.....	3.46	1.89	3.28	3.20	3.52	3.88	3.07	3.75	5.13	4.06	4.50	3.24	42.98	.....
Maximum....	8.35	3.57	7.19	4.43	5.08	6.38	6.92	6.70	7.73	7.47	10.22	4.11	54.83	.....
Minimum	0.55	0.93	1.41	2.21	0.59	2.59	1.72	1.78	3.55	2.35	1.54	1.98	30.58	.....

RUN-OFF.

1898-99.....	(2.34)	(1.21)	(1.80)	2.50	3.74	5.29	1.74	1.35	0.68	0.38	0.60	0.57	22.20	.....
1899-00.....	0.36	0.64	0.56	1.65	2.10	2.74	1.89	0.94	1.45	0.52	0.30	0.45	13.60	.....
1900-01.....	0.71	1.27	1.54	1.63	0.64	1.52	4.45	3.45	3.05	1.45	3.08	1.25	24.04	.....
1901-02.....	0.72	0.53	3.29	2.38	2.86	4.22	1.95	1.03	0.79	0.41	0.46	0.29	18.93	.....
1902-03.....	1.36	0.90	2.40	2.76	3.27	4.30	3.02	1.18	2.36	1.12	1.00	1.06	24.73	.....
1903-04.....	0.55	0.44	0.46	0.68	1.10	1.86	1.22	1.73	1.94	0.54	0.54	0.33	10.69	.....
1904-05.....	0.21	0.26	0.50	1.01	1.06	2.20	1.01	1.74	0.95	2.49	1.13	0.72	13.28	.....
Mean.....	0.89	0.75	1.51	1.80	2.11	3.16	2.18	1.63	1.50	0.99	1.02	0.67	18.21	.....
Maximum....	(2.34)	1.27	3.29	2.76	3.74	5.29	4.45	3.45	3.05	1.45	3.08	1.25	24.73	.....
Minimum	0.21	0.26	0.46	0.68	0.64	1.52	1.01	0.94	0.68	0.38	0.30	0.29	10.69	.....

RUN-OFF, IN PERCENTAGE OF PRECIPITATION.

1895–96	27	17	(18)	(36)	44	50	80	19	14	25	16	16	28
1896–97	14	26	34	28	92	80	67	50	17	16	16	22	50
1897–98	92	75	16	40	72	40	78	41	14	9	39	13	30
1898–99	29	53	44	75	76	102	92	40	18	8	5	9	45
1899–00	9	30	16	50	48	96	61	22	24	12	13	8	34
1900–01	25	59	67	64	105	58	76	56	52	31	31	26	49
1901–02	69	22	63	69	69	172	123	26	14	17	19	80	58
1902–03	8	14	63	55	72	84	83	79	24	38	15	33	46
1903–04	21	23	23	28	54	92	71	57	32	11	15	14	38
1904–05	41	19	11	24	16	129	40	35	21	36	18	14	33
Mean	24	29	43	49	64	86	76	42	25	22	22	16	41
Maximum	92	75	67	75	105	172	123	79	52	38	39	80	58
Minimum	8	14	11	24	16	40	40	19	14	8	5	8	28

LOSS, IN INCHES.

1895–96	0.64	1.54	(3.07)	(1.41)	2.27	2.62	0.31	2.45	2.49	4.96	2.95	5.21	31.92
1896–97	—0.20	3.83	—0.71	1.27	0.46	0.69	0.53	2.65	2.79	3.15	1.86	0.83	17.15
1897–98	2.17	2.45	3.11	1.90	0.24	2.26	0.64	3.37	3.01	4.36	3.56	2.84	29.91
1898–99	5.74	1.06	1.72	0.72	1.17	—0.11	0.14	2.78	2.82	2.66	4.03	3.33	26.06
1899–00	1.80	0.50	1.81	1.35	2.41	0.15	0.92	2.40	3.91	3.25	1.48	3.76	23.74
1900–01	2.85	1.71	0.86	0.91	—0.03	1.54	1.54	2.76	2.92	3.31	6.00	2.64	27.01
1901–02	0.20	1.07	2.81	0.94	1.62	—2.27	—0.37	1.87	3.17	1.89	1.30	2.66	14.89
1902–03	4.22	3.21	1.13	2.05	1.53	0.93	0.63	0.26	5.77	1.87	2.93	1.98	26.51
1903–04	1.55	0.89	1.04	1.68	0.90	0.19	0.67	1.62	3.65	3.54	1.98	1.29	19.00
1904–05	0.27	0.86	2.12	2.08	2.66	—0.69	1.30	4.10	3.52	5.46	2.51	2.22	26.41
Mean	1.92	1.71	1.70	1.43	1.32	0.53	0.64	2.43	3.60	3.44	2.86	2.67	24.26
Maximum	2.85	3.83	(3.07)	2.05	2.66	2.62	1.54	4.10	5.77	5.46	6.00	5.21	31.92
Minimum	—0.20	0.50	—0.71	0.72	—0.03	—2.27	—0.37	0.26	2.79	1.87	1.30	0.83	14.89

TABLE 2—(*Continued*).

JAMES RIVER, ABOVE BUCHANAN, VA. DRAINAGE AREA, 2 060 SQ. MILES.

PRECIPITATION.

Year.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Period.	No. Prec. Sta.
1895-96.....	0.88	1.85	3.76	2.20	4.06	5.26	1.57	3.03	5.19	6.60	3.52	6.20	44.12	6
1896-97.....	0.51	5.20	0.30	1.77	5.80	3.39	1.63	5.30	3.34	3.74	2.21	1.06	34.25	5
1897-98.....	2.39	2.65	3.70	3.18	0.86	3.79	2.94	5.76	3.50	4.81	5.83	3.28	42.69	7
1898-99.....	8.07	2.25	3.06	2.88	4.86	5.56	1.78	4.62	3.46	2.90	4.25	3.65	47.34	6
1899-00.....	1.98	0.71	2.16	2.70	4.61	3.52	2.37	3.10	5.14	3.69	1.71	4.02	35.78	5
1900-01.....	3.78	4.17	2.57	2.51	0.63	3.71	6.52	6.31	6.07	4.77	8.71	3.56	53.31	7
1901-02.....	0.64	1.37	7.63	3.03	5.26	3.16	1.64	2.53	3.69	2.27	1.61	2.89	35.72	5
1902-03.....	4.56	3.74	3.04	4.56	5.41	5.66	3.62	1.25	7.57	3.00	3.47	2.93	48.81	4
1903-04.....	1.95	1.16	1.36	2.34	1.94	2.39	2.31	3.81	5.40	3.96	2.33	1.50	30.45	5
1904-05.....	0.46	1.07	2.38	2.73	3.17	2.39	2.18	6.28	4.45	8.48	3.07	2.58	38.24	5
Mean.....	2.52	2.42	3.00	2.79	3.66	3.88	2.66	4.20	4.78	4.42	3.67	3.17	41.17	.....
Maximum...	8.07	5.20	7.63	4.56	5.80	5.66	6.52	6.31	7.57	8.48	8.71	6.20	53.31	.....
Minimum....	0.46	0.71	0.30	1.77	0.63	2.39	1.57	1.25	3.34	2.27	1.61	1.06	30.45	.....

RUN-OFF.

1895-96.....	0.24	0.31	(0.69)	(0.79)	1.79	2.64	1.26	0.58	0.70	1.64	0.57	0.99	12.20	.....
1896-97.....	0.71	1.37	1.01	0.50	5.34	2.70	1.10	2.65	6.55	0.59	0.35	0.23	17.10	.....
1897-98.....	0.22	0.20	0.59	1.28	0.62	1.53	2.30	2.39	0.49	0.45	2.27	0.44	12.78	.....
1898-99.....	2.33	1.19	1.34	2.16	3.69	5.67	1.64	1.84	0.64	0.24	0.22	0.32	21.28	.....
1899-00.....	0.18	0.21	0.35	1.35	2.20	3.37	1.45	0.70	1.23	0.44	0.23	0.33	12.04	.....
1900-01.....	0.93	2.46	1.71	1.60	0.66	2.17	4.98	3.55	3.15	1.46	2.71	0.92	26.30	.....
1901-02.....	0.44	0.30	4.82	2.09	3.64	5.43	2.01	0.66	0.52	0.38	0.31	0.23	20.83	.....
1902-03.....	0.34	0.53	1.91	2.51	3.88	4.73	2.99	0.99	1.80	1.13	0.54	0.95	22.30	.....
1903-04.....	0.40	0.27	0.32	0.66	1.04	2.20	1.64	2.19	1.75	0.42	0.35	0.21	11.45	.....
1904-05.....	0.19	0.21	0.26	0.65	0.51	3.08	0.88	2.18	0.93	3.02	0.56	0.36	12.83	.....
Mean.....	0.60	0.71	1.30	1.36	2.34	3.35	2.02	1.77	1.17	0.96	0.81	0.50	16.91	.....
Maximum...	2.33	2.46	4.82	2.51	5.34	5.67	4.98	3.55	3.15	3.02	2.71	0.99	26.30	.....
Minimum....	0.18	0.20	0.26	0.42	0.51	1.53	0.88	0.58	0.49	0.24	0.22	0.21	11.45	.....

Run-off, in Percentage of Precipitation.

1895–96.....	28	15	13	34	52	51	122	32	13	33	16	32	33	.....
1896–97.....	381	32	653	35	73	76	87	55	19	14	12	32	50	.....
1897–98.....	11	14	20	39	80	37	74	48	29	16	43	12	33	.....
1898–99.....	28	64	51	67	55	70	64	18	9	10	6	9	33	.....
1899–00.....	10	42	18	48	51	92	47	23	24	16	21	8	42	.....
1900–01.....	14	34	53	41	88	62	59	45	52	22	33	29	41	.....
1901–02.....	92	27	59	73	68	127	128	18	15	12	16	6	53	.....
1902–03.....	8	11	51	62	71	75	75	69	21	27	14	32	41	.....
1903–04.....	24	31	34	43	168	60	37	38	24	16	20	13	39	.....
1904–05.....	36	21	12	32	23	100	43	32	23	46	17	19	34	.....
Mean.....	26	28	42	49	66	70	66	37	24	24	22	19	39	.....
Maximum....	381	64	653	73	168	127	128	69	52	33	33	32	53	.....
Minimum....	8	11	12	34	51	37	37	18	9	10	6	6	33	.....

Loss, in Inches.

1895–96.....	0.66	1.60	3.08	1.88	2.02	2.55	—0.26	2.12	4.25	4.18	3.82	4.56	30.46	.....
1896–97.....	—0.90	3.60	—0.94	1.17	1.49	0.74	0.18	2.31	2.60	3.17	2.37	0.50	16.29	.....
1897–98.....	2.32	1.99	2.84	1.54	0.13	2.20	0.71	2.54	1.92	3.69	3.16	2.38	25.97	.....
1898–99.....	6.22	0.69	1.35	1.02	2.30	1.92	0.57	3.47	3.14	2.16	4.49	3.27	30.60	.....
1899–00.....	2.10	0.42	1.50	1.54	2.27	0.26	1.45	2.72	4.31	2.60	1.15	4.02	24.34	.....
1900–01.....	3.56	1.97	1.06	1.51	0.06	1.49	2.88	3.41	2.72	4.18	4.98	2.33	30.15	.....
1901–02.....	0.05	1.07	3.10	0.86	1.52	—0.86	—0.45	2.62	2.69	1.91	1.35	2.87	16.73	.....
1902–03.....	3.25	3.40	1.52	1.70	1.61	1.28	0.93	0.41	6.87	2.95	3.48	2.38	29.78	.....
1903–04.....	1.80	0.92	0.90	1.17	—1.34	1.12	2.03	2.61	4.05	2.84	2.01	1.59	19.70	.....
1904–05.....	0.39	0.92	2.25	2.10	2.23	—0.01	1.04	3.05	3.77	3.35	2.94	1.67	23.70	.....
Mean.....	1.94	1.66	1.67	1.45	1.23	1.07	0.91	2.53	3.63	3.10	2.97	2.61	24.77	.....
Maximum....	6.22	3.60	3.10	1.88	2.30	2.55	2.88	3.47	6.87	4.18	4.98	4.56	30.46	.....
Minimum....	—0.90	0.42	—0.94	0.86	—1.34	—0.86	—0.45	0.41	1.92	1.91	1.15	0.50	16.29	.....

TABLE 2—(*Continued*).

NORTH (OF JAMES) RIVER, ABOVE GLASGOW, VA. DRAINAGE AREA, 830 SQ. MILES.

PRECIPITATION.

Year.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Period.	No. Prec. Sta.
1895–96.....	0.92	1.88	3.54	2.86	4.17	5.18	1.19	3.10	4.89	6.22	4.57	6.70	45.22	2
1896–97.....	0.32	5.27	0.17	1.80	5.48	3.06	1.37	5.14	3.22	3.68	2.68	0.73	32.92	3
1897–98.....	2.60	2.31	3.56	2.53	0.66	3.48	2.71	4.92	2.72	4.41	5.54	3.34	38.78	4
1898–99.....	8.70	1.92	2.74	3.07	5.13	6.36	1.60	4.24	3.44	2.40	4.77	3.58	47.95	2
1899–00.....	2.34	0.73	1.83	2.95	4.59	3.17	2.74	3.54	5.67	3.08	1.46	4.39	36.49	4
1900–01.....	4.12	3.00	2.28	2.55	0.49	3.93	7.08	6.18	5.72	5.36	7.47	3.30	51.48	4
1901–02.....	0.60	1.46	7.60	3.17	4.81	3.14	1.59	3.20	3.15	2.18	1.60	3.04	35.54	4
1902–03.....	3.54	3.84	3.10	4.49	5.46	5.14	3.63	1.33	8.71	4.02	4.03	3.48	50.77	3
1903–04.....	2.38	1.33	1.36	2.07	1.97	2.81	3.22	4.24	5.36	3.40	2.52	1.82	32.48	3
1904–05.....	0.61	1.16	2.56	3.08	2.91	2.67	1.84	4.50	4.89	6.17	3.54	2.07	36.00	3
Mean.....	2.61	2.29	2.87	2.86	3.57	3.89	2.70	4.04	4.78	4.09	3.82	3.24	40.76	.....
Maximum....	8.70	5.27	7.60	4.49	5.48	6.36	7.08	6.18	8.71	6.22	7.47	6.70	51.48	.....
Minimum....	0.32	0.73	0.17	1.80	0.49	2.67	1.19	1.33	2.72	2.18	1.46	0.73	32.48	.....

RUN-OFF.

1895–96.....	0.26	0.28	0.46	0.98	2.15	2.63	1.45	0.98	0.64	2.04	0.75	2.14	14.76	.....
1896–97.....	1.22	1.67	1.11	0.63	3.99	2.32	1.19	2.83	0.62	0.51	0.31	0.23	16.63	.....
1897–98.....	0.28	0.32	0.72	0.99	0.53	1.28	2.00	2.38	0.80	0.72	2.38	0.41	12.81	.....
1898–99.....	2.48	1.23	1.39	2.05	2.83	4.44	1.03	0.77	0.30	0.24	0.28	0.31	17.35	.....
1899–00.....	0.24	0.31	0.33	1.41	2.32	2.91	1.29	0.82	1.36	0.48	0.31	0.37	12.15	.....
1900–01.....	0.56	1.03	1.22	1.04	0.43	2.44	4.20	2.77	3.00	1.18	2.49	0.97	21.33	.....
1901–02.....	0.55	0.39	4.50	2.31	3.29	4.00	2.04	0.58	0.46	0.27	0.25	0.17	18.81	.....
1902–03.....	0.29	0.44	1.58	2.79	3.85	3.86	2.70	0.92	1.84	1.07	0.55	1.10	20.99	.....
1903–04.....	0.58	0.41	0.46	0.90	3.31	1.69	1.19	1.63	1.31	0.56	0.51	0.23	12.78	.....
1904–05.....	0.22	0.24	0.31	0.98	0.68	2.68	0.80	1.45	1.12	2.82	0.60	0.40	12.30	.....
Mean.....	0.67	0.63	1.21	1.41	2.34	2.82	1.79	1.51	1.15	0.99	0.84	0.63	15.99	.....
Maximum....	2.48	1.67	4.50	2.79	3.99	4.44	4.20	2.83	3.00	2.82	2.49	2.14	21.33	.....
Minimum....	0.22	0.24	0.31	0.63	0.43	1.28	0.80	0.58	0.30	0.24	0.25	0.17	12.15	.....

RUN-OFF, IN PERCENTAGE OF PRECIPITATION.

1900-01.....	11	13	17	44	48	26	58	44	22	24	31	18	32	.....
1901-02.....	160	47	29	77	88	76	65	54	20	22	19	14	46	.....
1902-03.....	19	33	71	57	69	67	113	47	41	32	18	45	48	.....
1903-04.....	22	47	45	30	82	64	79	24	19	10	20	49	35	.....
1904-05.....	51	21	54	48	64	70	42	25	13	9	14	28	30	.....
Mean.....	23	29	39	53	74	62	69	36	23	18	23	29	39	.....
Maximum....	160	47	71	77	88	76	113	54	41	32	31	49	48	.....
Minimum....	11	13	17	30	48	26	42	24	13	9	14	14	30	.....

LOSS, IN INCHES.

1900-01.....	2.09	2.20	2.80	1.89	0.49	2.27	2.51	4.07	2.49	2.78	9.05	3.46	36.10	.....
1901-02.....	—0.21	0.56	5.77	0.76	0.46	0.98	0.82	0.79	4.04	1.51	2.88	2.26	20.62	.....
1902-03.....	6.16	2.67	1.06	2.09	1.62	1.95	—0.41	1.05	2.19	2.83	5.07	1.27	27.55	.....
1903-04.....	2.03	0.69	1.03	1.39	0.40	0.96	0.23	2.91	3.38	3.47	2.17	1.22	19.88	.....
1904-05.....	0.29	1.69	0.86	1.48	1.13	0.71	1.61	3.76	3.35	6.42	4.93	2.11	28.34	.....
Mean.....	2.07	1.56	2.30	1.52	0.82	1.38	0.95	2.52	3.09	3.40	4.82	2.07	26.50	.....
Maximum....	6.16	2.67	5.77	2.09	1.62	2.27	2.51	4.07	4.04	6.42	9.05	3.46	36.10	.....
Minimum....	—0.21	0.56	0.86	0.76	0.40	0.71	—0.41	0.79	2.19	1.51	2.17	1.22	19.88	.....

TABLE 2—(*Continued*).

APPOMATTOX RIVER, ABOVE MATTOAX, VA. DRAINAGE AREA, 745 SQ. MILES.

PRECIPITATION.

Year.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Period.	No. Prec. Sta.
1900–01.....	2.36	2.53	3.38	3.34	0.94	3.05	5.99	7.26	3.20	3.67	13.06	4.20	52.98	3
1901–02.....	0.35	1.06	8.08	3.37	3.96	4.09	2.33	1.72	5.04	1.94	3.56	2.63	38.13	2
1902–03.....	7.60	3.99	3.60	4.82	5.22	5.96	3.26	1.97	3.70	4.14	6.15	2.29	52.70	1
1903–04.....	2.61	1.31	1.88	2.00	2.27	2.68	1.08	3.83	4.18	3.86	2.70	2.40	30.80	2
1904–05.....	0.59	2.13	1.88	2.86	3.15	2.37	2.77	5.00	3.83	7.05	5.72	2.93	40.28	2
Mean.....	2.70	2.20	3.76	3.28	3.11	3.63	3.09	3.96	3.99	4.13	6.24	2.89	42.98	.....
Maximum....	7.60	3.99	8.08	4.82	5.22	5.96	5.99	7.26	5.04	7.05	13.06	4.20	52.98	.....
Minimum	0.35	1.06	1.88	2.00	0.94	2.37	1.08	1.72	3.20	1.94	2.70	2.29	30.80	.....

RUN-OFF.

1900–01.....	0.27	0.33	0.58	1.45	0.45	0.78	3.48	3.19	0.71	0.89	4.01	0.74	16.88	.....
1901–02.....	0.56	0.50	2.31	2.61	3.50	3.11	1.51	0.93	1.00	0.43	0.68	0.37	17.51	.....
1902–03.....	1.44	1.32	2.54	2.73	3.60	4.01	3.67	0.92	1.51	1.31	1.08	1.02	25.15	.....
1903–04.....	0.58	0.62	0.85	0.61	1.87	1.72	0.85	0.92	0.80	0.39	0.53	1.18	10.92	.....
1904–05.....	0.30	0.44	1.02	1.38	2.02	1.66	1.16	1.24	0.48	0.63	0.79	0.82	11.94	.....
Mean.....	0.63	0.64	1.46	1.75	2.29	2.26	2.13	1.44	0.90	0.73	1.42	0.83	16.48	.....
Maximum....	1.44	1.32	2.54	2.73	3.60	4.01	3.67	3.19	1.51	1.31	4.01	1.18	25.15	.....
Minimum	0.27	0.33	0.58	0.61	0.45	0.78	0.85	0.92	0.48	0.39	0.53	0.37	10.92	.....

Run-off, in Percentage of Precipitation.

1896–97.....	44	27	92	13	57	63	69	62	57	13	17	26	42	.....
1897–98.....	19	21	10	47	152	27	(57)	36	28	14	47	29	32	.....
1898–99.....	48	77	75	90	96	115	49	35	16	12	8	9	56	.....
1899–00.....	16	23	27	46	45	58	81	27	21	21	17	7	32	.....
1900–01.....	27	44	52	63	93	48	75	58	36	48	53	37	51	.....
1901–02.....	92	51	57	78	76	102	89	41	14	13	27	9	54	.....
1902–03.....	9	15	7	(46)	(55)	70	87	125	22	24	21	34	43	.....
1903–04.....	29	25	30	22	33	48	25	25	15	15	41	31	25	.....
1904–05.....	208	17	11	19	15	82	49	33	34	30	28	42	31	.....
Mean.....	31	31	39	50	60	72	66	43	24	24	35	24	41	.....
Maximum...	208	77	92	90	152	115	89	125	57	48	53	37	56	.....
Minimum...	9	15	7	13	15	27	25	25	14	12	8	7	25	.....

Loss, in Inches.

1896–97.....	0.53	3.25	0.04	1.42	3.04	1.38	0.51	1.42	0.81	4.12	2.36	1.41	20.29	.....
1897–98.....	3.12	0.89	3.33	1.55	—0.32	2.53	(0.84)	3.92	2.04	3.72	1.89	2.65	26.16	.....
1898–99.....	3.26	0.43	0.64	0.39	0.20	—1.00	1.44	2.11	4.23	2.77	3.41	3.65	21.53	.....
1899–00.....	1.33	1.25	1.41	1.42	2.12	2.05	0.46	2.60	4.63	2.44	1.24	4.81	25.76	.....
1900–01.....	3.13	2.17	1.60	0.87	0.05	1.68	1.60	3.10	4.52	2.71	4.99	2.22	28.64	.....
1901–02.....	0.07	0.51	3.22	0.64	(1.23	—0.06	0.20	1.09	3.30	2.81	0.72	2.18	15.91	.....
1902–03.....	3.96	3.91	0.75	(2.22)	2.41)	1.92	0.51	—0.23	4.43	2.81	2.86	3.03	28.58	.....
1903–04.....	1.53	1.44	0.87	1.63	1.26	1.26	1.70	2.57	6.95	4.32	1.96	0.84	26.33	.....
1904–05.....	—0.14	1.55	2.81	2.61	4.10	0.41	0.93	4.91	1.75	8.10	2.81	1.87	31.71	.....
Mean.....	1.86	1.71	1.63	1.42	1.57	1.13	0.94	2.39	3.63	3.75	2.47	2.53	24.99	.....
Maximum...	3.96	3.91	3.33	2.61	4.10	2.53	1.70	4.91	6.95	8.10	4.99	4.81	31.71	.....
Minimum....	—0.14	0.43	0.04	0.39	—0.32	—1.00	0.20	—0.23	0.81	2.44	0.72	0.84	15.91	.....

TABLE 2—(*Continued*).

ROANOKE RIVER, ABOVE ROANOKE, VA. DRAINAGE AREA, 390 SQ. MILES.

PRECIPITATION.

Year.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Period.	No. Prec. Sta.
1896-97.....	0.95	4.47	0.50	1.64	7.10	3.74	1.67	3.71	1.90	4.72	2.88	1.96	35.19	3
1897-98.....	3.86	1.13	3.72	2.90	0.62	3.47	1.96	6.14	2.82	4.32	3.59	3.75	38.28	3
1898-99.....	6.32	1.89	2.60	3.73	5.83	6.49	2.82	3.23	5.03	3.16	3.72	4.03	48.85	3
1899-90.....	1.59	1.62	1.92	2.62	3.84	4.91	2.44	3.55	5.85	3.08	1.50	5.16	38.08	3
1900-01.....	4.26	3.88	3.31	2.36	0.71	3.26	6.50	7.46	7.06	5.26	10.72	3.52	58.30	3
1901-02.....	0.86	1.04	7.45	2.96	5.05	3.04	1.76	1.85	3.84	3.24	0.98	2.40	34.47	2
1902-03.....	4.36	4.60	2.43	4.08	5.40	6.46	3.94	0.93	5.65	3.72	3.60	4.56	49.73	1
1903-04.....	2.14	1.91	1.25	2.08	1.90	2.44	2.28	3.44	8.14	5.10	3.31	1.22	35.21	2
1904-05.....	0.13	1.87	3.16	3.24	4.80	2.25	1.84	7.30	2.66	11.64	3.92	3.22	46.08	1
Mean.....	2.71	2.49	2.92	2.85	3.92	4.01	2.80	4.18	4.77	4.91	3.80	3.32	42.68	.....
Maximum....	6.32	4.60	7.45	4.08	7.10	6.49	6.50	7.46	8.14	11.64	10.72	5.16	58.30	.....
Minimum	0.13	1.04	0.50	1.64	0.62	2.44	1.67	0.93	1.90	3.08	0.98	1.22	34.47	.....

RUN-OFF.

1896-97..	0.42	1.22	0.46	0.22	4.06	2.36	1.16	2.29	1.09	0.60	0.47	0.55	14.90	.....
1897-98.....	0.74	0.24	0.39	1.35	0.94	0.94	(1.12)	2.22	0.78	0.60	1.70	1.10	12.12	....
1898-99.....	3.06	1.46	1.96	3.34	5.63	7.49	1.38	1.12	0.80	0.39	0.31	0.38	27.32	.....
1899-00.....	0.26	0.37	0.51	1.20	1.72	2.86	1.98	0.95	1.22	0.64	0.26	0.35	12.32	.....
1900-01..	1.13	1.71	1.71	1.49	0.66	1.58	4.90	4.36	2.54	2.55	5.73	1.30	29.66	.....
1901-02.....	0.79	0.53	4.23	2.32	3.82	3.10	1.56	0.76	0.54	0.43	0.26	0.22	18.56	.....
1902-03.....	0.40	0.69	1.68	(1.86)	(2.99)	4.54	3.43	1.16	1.22	0.91	0.74	1.53	21.15	.....
1903-04.....	0.61	0.47	0.38	0.45	0.64	1.18	0.58	0.87	1.19	0.78	1.35	0.38	8.88	.....
1904-05.....	0.27	0.32	0.35	0.63	0.70	1.84	0.91	2.39	0.91	3.54	1.11	1.35	14.32	.....
Mean.....	0.85	0.78	1.29	1.43	2.35	2.88	1.89	1.79	1.14	1.16	1.33	0.80	17.69	.....
Maximum....	3.06	1.71	4.23	3.34	5.63	7.49	4.90	4.36	2.54	3.54	5.73	1.53	29.66	.....
Minimum	0.26	0.24	0.35	0.22	0.64	0.94	0.58	0.76	0.54	0.39	0.26	0.22	8.88	.....

RUN-OFF, IN PERCENTAGE OF PRECIPITATION.

1900–01.....	25	34	44	71	116	44	58	50	38	46	44	44	47	.....
1901–02.....	180	89	46	63	67	103	87	54	33	30	34	25	57	.....
1902–03.....	33	30	53	53	51	64	58	66	28	28	24	40	44	.....
1903–04.....	35	39	41	36	58	41	55	32	21	16	30	35	32	.....
1904–05.....	45	17	24	36	(31)	76	31	23	37	28	27	37	32	.....
Mean.....	40	36	43	53	56	65	56	41	30	29	35	37	43	.....
Maximum....	180	89	53	71	116	103	87	66	38	46	44	44	57	.....
Minimum....	25	17	24	36	(31)	41	31	23	21	16	24	25	32	.....

LOSS, IN INCHES.

1900–01.....	2.84	2.08	1.92	0.82	−0.14	1.91	2.55	3.17	2.78	2.75	6.27	1.84	28.79	.....
1901–02.....	−0.56	0.14	4.23	1.18	1.75	−0.09	0.24	1.02	2.65	1.87	1.58	1.99	16.00	.....
1902–03.....	3.72	2.13	1.76	2.06	2.73	2.36	1.86	0.65	3.86	2.38	3.24	1.74	28.49	.....
1903–04.....	1.53	1.06	1.05	1.41	0.93	1.47	0.67	2.32	4.75	4.04	2.57	1.21	23.01	.....
1904–05.....	0.35	1.63	2.23	1.86	(2.65)	0.56	2.03	4.95	1.78	6.30	3.08	1.96	29.38	.....
Mean.....	0.58	1.41	2.24	1.47	1.58	1.24	1.47	2.42	3.16	3.47	3.35	1.75	25.14	.....
Maximum....	3.72	2.13	4.23	2.06	2.73	2.36	2.55	4.95	4.75	6.30	6.27	1.99	29.38	.....
Minimum....	−0.56	0.14	1.05	0.82	−0.14	−0.09	0.24	0.65	1.78	1.87	1.58	1.21	16.00	.....

TABLE 2—(*Continued*).

ROANOKE RIVER, ABOVE RANDOLPH, VA. DRAINAGE AREA, 3 080 SQ. MILES.

PRECIPITATION.

Year.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Period.	No. Prec. Sta.
1900-01.....	3 81	3.15	3.40	2.83	0.88	3.41	6.04	6.33	4.51	5.09	11.21	3.29	53.95	6
1901-02.....	0.70	1.22	7.84	3.20	5.24	3.14	1.84	2.24	3.98	2.68	2.40	2.66	37.14	4
1902-03.....	5.54	3.05	3.77	4.35	5.63	6.49	4.41	1.92	5.37	3.29	4.25	2.92	50.99	4
1903-04.....	2.35	1.73	1.78	2.19	2.21	2.51	1.48	3.42	5.98	4.83	3.66	1.86	34.00	7
1904-05.....	0.65	1.95	2.94	2.90	3.85	2.29	2.96	6.44	2.83	8.73	4.23	3.12	42.89	7
Mean.....	2.61	2.22	3.95	3.10	3.56	3.57	3.35	4.07	4.53	4.92	5.15	2.77	43.80	.....
Maximum....	5.54	3.15	7.84	4.35	5.63	6.49	6.04	6.33	5.98	5.09	11.21	3.29	53.95	.....
Minimum....	0 65	1.22	1.78	2.19	0.88	2.29	1.48	1.92	2.83	2.68	2.40	1.86	34.00	.....

RUN-OFF.

1900-01.....	0.97	1.07	1.48	2.01	1.02	1.50	3.49	3.16	1.73	2.34	4.94	1.45	25.16	.....
1901-02.....	1.26	1.08	3.61	2.02	3.49	3.23	1.60	1.22	1.33	0.81	0.82	0.67	21.14	.....
1902-03.....	1.82	0.92	2.01	2.29	2.90	4.13	2.55	1.27	1.51	0.91	1.01	1.18	22.50	.....
1903-04.....	0.82	0.67	0.73	0.78	1.28	1.04	0.81	1.10	1.23	0.79	1.09	0.65	10.99	.....
1904-05.....	0.30	0.32	0.71	1.04	(1.20)	1.73	0.93	1.49	1.05	2.43	1.15	1.16	13.51	.....
Mean.....	1.03	0.81	1.71	1.63	1.98	2.33	1.88	1.65	1.37	1.45	1.80	1.02	18.66	.....
Maximum....	1.82	1.08	3.61	2.29	3.49	4.13	3.49	3.16	1.73	2.43	4.94	1.45	25.16	.....
Minimum....	0.30	0.32	0.71	0.78	1.02	1.04	0.81	1.10	1.05	0.79	0.82	0.65	10.99	.....

TABLE 3.—MONTHLY AND YEARLY MAXIMUM, MINIMUM AND MEAN LOSS, FOR AN

Note:—M. = Mean;

PRECIPITATION, IN INCHES.

Drainage.	OCTOBER.		NOVEMBER.		DECEMBER.		JANUARY.		FEBRUARY.		MARCH.	
	M.	R.	M.	R.	M.	R.	M.	R.	M.	R.	M.	R.
Connecticut, above Orford; 3 300 sq. miles....	2.99	4.12 2.12	2.17	5.54 1.05	2.80	4.77 1.36	2.28	2.94 1.99	1.60	3.18 0.71	3.44	4.77 2.05
Housatonic, above Gaylordsville; 1020 sq. miles	3.98	6.49 2.74	2.39	4.29 0.89	4.38	6.76 2.72	3.05	4.89 1.65	2.56	4.24 0.76	4.10	5.20 3.04
Susquehanna, above Harrisburg; 24 030 sq. miles.	3.02	5.74 0.95	2.63	4.43 0.92	2.97	5.63 1.04	2.73	4.40 1.77	2.67	4.55 0.93	3.35	4.58 1.21
Susquehanna, above Wilkes-Barre; 9810 sq. miles.....	3.33	6.04 1.69	2.41	4.70 1.13	3.30	5.58 2.24	2.57	3.40 1.69	2.30	3.46 1.17	3.60	4.77 3.17
Susquehanna, above Williamsport; 5 640 sq. miles	2.90	6.22 0.89	2.74	4.91 0.54	3.15	5.48 1.25	2.63	3.69 1.51	2.72	4.00 1.05	4.09	5.20 3.42
Ohio, above Wheeling; 23 820 sq. miles.....	2.66	6.78 0.51	3.09	5.67 0.65	3.13	5.07 1.84	3.24	4.97 1.72	3.19	6.54 1.19	3.39	5.62 1.40
Potomac, above Point of Rocks; 9 650 sq. miles..	2.21	6.41 0.57	2.29	4.10 0.79	2.67	5.71 0.74	2.53	3.78 1.55	2.91	5.88 0.46	3.42	4.44 2.08
Shenandoah, above Millville; 3 000 sq. miles....	2.47	7.10 0.48	2.08	4.16 0.81	2.61	6.12 0.23	2.58	4.08 1.41	3.12	6.38 0.33	3.52	5.12 2.08
James, above Cartersville; 6 230 sq. miles.....	3.46	8.35 0.55	1.89	3.57 0.93	3.28	7.19 1.41	3.20	4.43 2.21	3.52	5.08 0.59	3.88	6.38 2.59
James, above Buchanan; 2 060 sq. miles.....	2.52	8.07 0.46	2.42	5.20 0.71	3.00	7.63 0.30	2.79	4.56 1.77	3.66	5.80 0.63	3.88	5.66 2.39
James, above Glasgow; 830 sq. miles.....	2.61	8.70 0.32	2.29	5.27 0.73	2.87	7.60 0.17	2.86	4.49 1.80	3.57	5.48 0.49	3.89	6.36 2.67
Appomattox, above Mattoax; 745 sq. miles.....	2.70	7.60 0.35	2.20	3.99 1.06	3.76	8.08 1.88	3.28	4.82 2.00	3.11	5.22 0.94	3.63	5.96 2.37
Roanoke, above Roanoke; 390 sq. miles.....	2.71	6.32 0.13	2.49	4.60 1.04	2.92	7.45 0.50	2.85	4.08 1.64	3.92	7.10 0.62	4.0	6.49 2.44
Roanoke, above Randolph; 3 080 sq. miles...	2.61	5.54 0.65	2.22	3.15 1.22	3.95	7.84 1.78	3.10	4.35 2.19	3.56	5.63 0.88	3.57	6.49 2.29

RUN-OFF, IN INCHES.

Drainage.	OCTOBER.		NOVEMBER.		DECEMBER.		JANUARY.		FEBRUARY.		MARCH.	
	M.	R.	M.	R.	M.	R.	M.	R.	M.	R.	M.	R.
Connecticut, above Orford.....	1.24	1.94 0.45	1.23	2.51 0.50	1.28	2.34 0.47	0.76	1.11 0.27	0.54	1.03 0.26	3.91	7.56 1.60
Housatonic, above Gaylordsville	1.89	3.25 (0.40)	1.39	1.92 0.96	2.68	4.13 (1.03)	2.32	3.21 0.98	1.61	(3.68) 0.49	5.88	8.77 (4.24)
Susquehanna, above Harrisburg	0.90	2.17 0.16	1.08	2.18 0.28	1.75	3.53 0.49	1.94	3.79 0.67	1.98	4.04 0.53	4.48	7.46 2.46
Susquehanna, above Wilkes-Barre.....	1.16	3.22 0.13	0.96	1.47 0.60	2.53	4.91 0.90	2.89	3.45 2.14	2.56	3.92 1.57	5.33	7.84 2.79
Susquehanna, above Williamsport.....	0.85	2.68 0.15	1.14	1.84 0.29	1.75	4.14 0.33	1.96	3.23 1.01	1.96	4.52 0.56	5.59	8.09 2.82
Ohio, above Wheeling....	0.72	3.70 0.12	1.21	2.97 0.24	1.99	3.64 0.53	2.78	4.30 1.28	3.12	7.29 0.78	4.07	6.89 1.37
Potomac, above Point of Rocks.....	0.52	1.63 0.14	0.44	0.99 0.15	1.06	3.06 0.26	1.30	2.49 0.51	2.00	4.60 0.39	2.80	6.50 1.34
Shenandoah, above Millville.....	0.82	2.99 0.20	0.48	1.05 0.20	1 05	3.12 0.29	1.21	2.62 0.47	1.53	3.63 0.46	2.16	5.34 0.86
James, above Cartersville.....	0.89	2.34 0.21	0.75	1.27 0.26	1.51	3.29 0.46	1.80	2.76 0.68	2.11	3.74 0.64	3.16	5.29 1.52
James, above Buchanan.	0.60	2.33 0.18	0.71	2.46 0.20	1.30	4.82 0.26	1.36	2.51 0.42	2.34	5.34 0.51	3.35	5.67 1.53
James, above Glasgow...	0.67	2.48 0.22	0.63	1.67 0.24	1.21	4.50 0.31	1.41	2.79 0.63	2.34	3.99 0.43	2.82	4.44 1.28
Appomattox, above Mattoax.....	0.63	1.44 0.27	0.64	1.32 0.33	1.46	2.54 0.58	1.75	2.73 0.61	2.29	3.60 0.45	2.26	4.01 0.78
Roanoke, above Roanoke.	0.85	3.06 0.26	0.78	1.71 0.24	1.29	4.23 0.35	1.43	3.34 0.22	2.34	5.63 0.64	2.88	7.49 0.94
Roanoke, above Randolph.....	1.08	1.82 0.30	0.81	1.08 0.82	1.71	3.61 0.71	1.63	2.29 0.78	1.98	3.49 1.02	2.33	4.13 1.04

RAINFALL, RUN-OFF, RUN-OFF IN PERCENTAGE OF RAINFALL, AND AVERAGE YEAR.
R. = Range.

APRIL.		MAY.		JUNE.		JULY.		AUGUST.		SEPTEMBER.		YEAR.		No. of Years.
M.	R.	M.	R.	M.	R.	M.	R.	M.	R.	M.	R.	Total.	R.	
	3.54		4.78		5.27		5.06		4.59		5.75		41.80	
2.77	1.37	2.99	0.27	3 78	2.10	4.34	3.76	3.88	3.22	3.73	1.08	36.76	33.48	5
	6.12		6.24		10.42		7.25		7.28		6.42		51.49	
3.71	2.25	2.97	1.12	5.46	1.86	5.00	3.72	5.56	3.45	4.70	1.94	47.86	39.77	5
	4.46		7.70		6.44		7.24		6.48		5.61		45.17	
2.76	1.27	3.96	1.27	3.98	2.77	4.11	2.42	4.16	1.92	3.04	1.41	39.38	31.62	14
	4.67		5.39		6.38		7.86		6.51		4.32		44.13	
2.70	1.59	2.73	1.11	4.46	2.94	5.05	4.03	4.49	2.73	2.90	1.40	39.85	31.77	6
	4.69		5.41		6.03		7.58		6.62		4.70		44.11	
2.89	1.33	3.20	1.74	4.11	2.94	4.62	2.77	4.14	2.26	2.83	1.05	40.62	33.04	10
	6.50		7.48		5.80		9.08		6.88		6.48		55.56	
3.28	1.57	4.04	2.18	4.32	2.50	4.55	2.54	3.74	1.30	3.07	1.56	41.71	33.47	21
	3.05		6.47		6.57		6.63		7.00		6.09		44.81	
2.61	1.34	3.77	1.97	4.15	1.81	4.15	2.28	3.50	1.69	2.65	1.32	36.86	29.37	10
	6.24		5.82		7.63		6.21		7.73		7.22		48.08	
2.55	1.16	3.85	2.28	4.90	2.09	4.14	2.17	3.56	1.41	2.95	1.01	38.33	30.47	10
	6.92		6.70		7.73		7.47		10.22		4.11		54.83	
3.07	1.72	3.75	1.78	5.13	3.55	4.06	2.35	4.50	1.54	3.24	1.98	42.98	30.58	7
	6.52		6.31		7.57		8.48		8.71		6.20		53.31	
2.66	1.57	4.20	1.25	4.78	3.34	4.42	2.27	3.67	1.61	3.17	1.06	41.17	30.45	10
	7.08		6.18		8.71		6.22		7.47		6.70		51.48	
2.70	1.19	4.04	1.33	4.78	2.72	4.09	2.18	3.82	1.46	3.24	0.73	40.76	32.48	10
	5.99		7.26		5.04		7.05		13.06		4.20		52.98	
3.09	1.08	3.96	1.72	3.99	3.20	4.13	1.94	6.24	2.70	2.89	2.29	42.98	30.80	5
	5.50		7.46		8.14		11.64		10.72		5.16		58.30	
2.80	1.67	4.18	0.93	4.77	1.90	4.91	3.08	3.80	0.98	3.32	1.22	42.68	35.19	9
	6.04		6.33		5.98		5.09		11.21		3.20		53.95	
3.35	1.48	4.07	1.92	4.53	2.83	4.92	2.68	5.15	2.40	2.77	1.86	43.80	34.00	5

APRIL.		MAY.		JUNE.		JULY.		AUGUST.		SEPTEMBER.		YEAR.		No. of Years.
M.	R.	L.	R.	M.	R.	M.	R.	M.	R.	M.	R	Total.	R.	
	7.10		4.80		3.20		1.51		1.53		1.82		27.04	
4.70	3.64	3.10	1.16	1.69	1.02	1.09	0.49	1.09	0.69	1.03	0.37	21.66	16.01	5
	6.43		4.59		4.28		2.49		2.19		2.25		36.94	
4.63	3.82	2.40	1.11	2.24	0.92	1.47	0.56	1.41	0.87	1.51	1.08	29.43	23.76	5
	4.83		4.54		3.03		3.25		1.60		1.42		28.03	
3.43	2.34	2.07	0.61	1.25	0.50	0.83	0.34	0.77	0.24	0.61	0.17	21.09	16.34	14
	4.46		2.52		1.79		3.41		1.53		1.44		27.18	
3.17	2.49	1.13	0.40	1.07	0.40	1.01	0.28	0.66	0.12	0.72	0.15	23.19	15.15	6
	5.45		3.15		2.44		4.11		1.44		1.24		27.60	
3.50	2.24	1.68	0.60	1.20	0.54	1.23	0.36	0.87	0.27	0.52	0.18	22.26	16.57	10
	6.84		5.10		3.37		3.49		1.88		2.50		34.20	
3.20	1.80	1.94	0.51	1.30	0.31	1.06	0.28	0.76	0.16	0.53	0.15	22.68	16.29	21
	4.60		3.22		2.18		1.52		2.66		0.88		21.46	
1.98	0.76	1.34	(0.31)	0.99	0.37	0.76	0.29	0.69	0.23	0.34	0.16	14.22	8.16	10
	4.79		3.36		3.07		1.71		3.15		0.53		19.78	
1.77	0.72	1.39	0.53	1.16	0.52	0.83	0.34	0.81	0.38	0.43	0.23	13.64	7.86	10
	4.45		3.45		3.05		1.45		3.08		1.25		24.73	
2.18	1.01	1.63	0.94	1.50	0.68	0.99	0.38	1.02	0.30	0.67	0.39	18.21	10.69	7
	4.98		3.55		3.15		3.02		2.71		0.99		26.30	
2.02	0.88	1.77	0.58	1.17	0.49	0.98	0.24	0.81	0.22	0.50	0.21	16.91	11.45	10
	4.20		2.83		3.00		2.82		2.49		2.14		21.33	
1.79	0.80	1.51	0.58	1.15	0.30	0.99	0.24	0.84	0.25	0.63	0.17	15.99	12.15	10
	3.67		3.19		1.51		1.31		4.01		1.18		25.15	
2.13	0.85	1.44	0.92	0.90	0.48	0.73	0.39	1.42	0.53	0.83	0.37	16.48	10.92	5
	4.90		4.36		2.54		3.54		5.73		1.53		29.66	
1.89	0.58	1.79	0.76	1.14	0.54	1.16	0.39	1.33	0.26	0.80	0.22	17.69	8.88	9
	3.49		3.16		1.73		2.43		4.94		1.45		25.18	
1.88	0.81	1.65	1.10	1.37	1.05	1.45	0.79	1.80	0.82	1.02	0.65	18.66	10.99	5

TABLE 3—

RUN-OFF, IN PERCENTAGE OF RAINFALL.

Drainage.	October.		November.		December.		January.		February.		March.	
	M.	R.	M.	R.	M.	R.	M.	R.	M.	R.	M.	R.
Connecticut, above Orford	41	92 17	57	132 33	46	103 17	33	55 13	34	86 23	114	163 45
Housatonic, above Gaylordsville	47	64 (13)	58	175 22	61	67 (37)	76	149 (41)	63	(87) (89)	143	208 87
Susquehanna, above Harrisburg	30	44 6	41	105 11	59	100 19	71	133 32	74	209 41	134	277 60
Susquehanna, above Wilkes-Barre	35	53 4	40	85 22	77	140 40	112	201 76	111	197 (47)	148	223 78
Susquehanna, above Williamsport	29	64 10	42	68 12	56	102 15	75	116 50	72	146 43	137	200 74
Ohio, above Wheeling	27	72 6	39	114 12	64	99 27	86	129 46	98	151 54	120	191 82
Potomac, above Point of Rocks	24	162 8	19	41 6	40	76 10	51	98 30	69	163 33	82	149 41
Shenandoah, above Millville	33	623 10	23	64 12	40	383 9	47	78 (27)	49	139 (26)	61	148 23
James, above Cartersville	25	131 13	36	69 23	46	70 21	56	74 31	60	108 34	81	128 44
James, above Buchanan	24	92 8	29	75 14	43	67 11	49	75 24	64	105 16	86	172 40
James, above Glasgow	26	381 8	28	64 11	42	653 12	49	73 34	66	168 51	70	127 37
Appomattox, above Mattoax	23	160 11	29	47 13	39	71 17	53	77 30	74	88 48	62	76 26
Roanoke, above Roanoke	31	208 9	31	77 15	39	92 7	50	90 13	60	152 15	72	115 27
Roanoke, above Randolph	40	180 25	36	89 17	43	53 24	53	71 36	56	116 (31)	65	103 41

LOSS, IN INCHES.

Drainage.	October.		November.		Decemeber.		January.		February.		March.	
	M.	R.	M.	R.	M.	R.	M.	R.	M.	R.	M.	R.
Connecticut, above Orford	1.75	2.60 0.18	0.94	3.03 —0.41	1.52	2.43 —0.06	1.52	1.97 0.91	1.06	2.15 0.10	—0.47	1.92 2.91
Housatonic, above Gaylordsville	2.09	3.24 0.98	1.00	3.33 —0.67	1.70	2.63 0.96	0.73	1.99 —1.05	0.95	2.36 (0.20)	—1.78	0.66 —4.46
Susquehanna, above Harrisburg	2.12	4.16 0.66	1.55	3.42 —0.11	1.22	2.76 0.00	0.79	1.84 —0.62	0.69	2.14 —1.21	—1.13	1.65 —3.58
Susquehanna, above Wilkes-Barre	2.17	3.45 1.27	1.45	3.46 0.17	0.77	1.70 —0.93	—0.32	0.83 —1.72	—0.26	0.65 —1.93	—1.73	0.80 —4.83
Susquehanna, above Williamsport	2.05	4.71 0.46	1.60	3.58 0.21	1.40	2.82 —0.03	0.67	1.50 —0.39	0.76	1.92 —0.84	—1.50	1.02 —4.05
Ohio, above Wheeling	1.94	3.96 0.39	1.88	4.76 —0.37	1.14	2.35 0.03	0.46	1.91 —0.88	0.07	0.90 —1.28	—0.68	0.69 —3.13
Potomac, above Point of Rocks	1.68	4.78 0.57	1.85	3.86 0.64	1.61	2.65 0.18	1.23	2.09 0.05	0.91	2.32 —0.75	0.62	2.60 —2.15
Shenandoah, above Millville	1.65	4.91 —2.51	1.60	3.48 0.29	1.56	3.00 —0.65	1.37	2.11 0.55	1.59	(3.67) —0.13	1.36	(2.40) —1.73
James, above Cartersville	2.57	(6.01) —0.17	1.14	2.67 0.29	1.77	3.90 0.95	1.40	1.93 0.83	1.41	2.31 —0.05	0.72	1.96 —0.92
James, above Buchanan	1.92	2.85 —0.20	1.71	3.83 0.50	1.70	3.07 —0.71	1.43	2.05 0.72	1.32	2.66 —0.03	0.53	2.62 —2.27
James, above Glasgow	1.94	6.22 —0.90	1.66	3.60 0.42	1.67	3.19 —0.94	1.45	1.88 0.86	1.23	2.30 —1.34	1.07	2.55 —0.86
Appomattox, above Mattoax	2.07	6.16 —0.21	1.56	2.67 0.56	2.30	5.77 0.86	1.52	2.09 0.76	0.82	1.62 0.40	1.38	2.27 0.71
Roanoke, above Roanoke	1.86	3.96 —0.14	1.71	3.91 0.43	1.63	3.33 0.04	1.42	2.61 0.39	1.57	4.19 —0.32	1.13	2.53 1.00
Roanoke, above Randolph	1.58	3.72 —0.56	1.41	2.13 0.14	2.24	4.23 1.05		2.06 0.82	1.58	2.73 —0.14	1.24	2.36 —0.09

(Continued).

APRIL.		MAY.		JUNE.		JULY.		AUGUST.		SEPTEMBER.		YEAR.		No. of Years.
M.	R.	M.	R.	M.	R.	M.	R.	M.	R.	M.	R.	Mean.	R.	
	266		430		68		38		39		34		65	
170	120	104	65	45	20	25	13	28	16	28	22	59	46	5
	187		118		113		61		40		86		73	
125	105	81	72	41	20	29	11	25	18	32	17	62	53	5
	324		72		86		47		47		73		63	
124	25	52	21	31	10	20	9	18	6	20	5	55	44	14
	200		50		58		43		6		74		65	
117	96	41	26	24	10	20	7	15	4	25	11	58	47	6
	276		69		66		54		82		56		63	
121	85	52	34	29	12	27	12	21	7	18	7	56	42	10
	156		82		67		53		56		39		63	
98	58	48	18	30	10	23	9	20	5	17	6	54	44	21
	104		57		48		34		38		28		61	
76	35	36	(11)	24	11	18	9	20	10	13	4	39	22	10
	113		65		41		43		41		41		58	
69	31	36	14	24	13	20	11	23	10	15	5	36	21	10
	101		66		54		37		30		44		50	
71	48	43	32	29	18	24	12	23	11	21	8	42	33	7
	123		79		52		38		39		80		58	
76	40	42	19	25	14	22	8	22	5	16	8	41	28	10
	128		69		52		33		33		32		53	
66	37	37	18	24	9	24	10	22	6	19	6	39	33	10
	113		54		41		32		31		49		48	
69	42	36	24	23	13	18	9	23	14	29	14	38	30	5
	89		125		57		48		53		37		56	
66	25	43	75	24	14	24	12	35	8	24	7	41	25	9
	87		66		38		46		44		44		57	
56	31	41	23	20	21	29	16	35	24	37	25	43	32	5

APRIL.		MAY.		JUNE.		JULY.		AUGUST.		SEPTEMBER.		YEAR.		No. of Years.
M.	R.	M.	R.	M.	R.	M.	R.	M.	R.	M.	R.	Total.	R.	
	−0.70		1.67		4.23		3.81		3.62		3.74		18.81	
−1.94	−3.62	−0.11	−1.39	09	1.05	3.25	2.41	2.79	2.00	2.70	0.71	15.10	12.84	5
	−0.20		1.74		6.14		5.06		5.71		5.19		22.56	
−0.92	−2.20	0.57	−0.22	.22	−0.25	3.53	1.56	4.15	2.08	3.19	0.39	18.43	13.39	5
	1.37		3.16		5.57		4.41		5.25		4.81		21.04	
−0.67	−2.84	1.9	0.58	2.3	0.40	3.28	1.94	3.39	1.22	2.43	0.53	18.29	13.54	14
	0.21		2.87		5.30		4.53		5.10		3.76		18.61	
−0.46	−1 50	1.60	0.71	3.39	1.32	4.04	3.47	3.83	2.53	2.18	0.43	16.66	14.52	6
	0.47		2.26		5.05		4.58		5.18		4.39		20.89	
−0.61	−2 54	1.52	0.94	2.91	1.25	3.39	2.41	3.27	0.96	2.31	0.87	17.76	15.70	10
	2.00		3.14		4.91		6.07		5.44		4.50		24.86	
0.07	−1.56	2.10	0.95	3.02	1.33	3.50	2.26	2.98	1.09	2.53	0.81	19.02	15.18	21
	1.60		3.77		4.49		5.41		5.23		5.84		29.09	
0.63	−0.14	2.43	1.26	3.16	1.33	3.39	1.98	2.81	1.27	2 31	1.03	22.64	13.87	10
	1.82		3.97		5.14		5.11		4.58		6.83		33.05	
0.78	−0.30	2.46	1.46	3.74	1.49	3.31	1.73	2.75	1.03	2.52	0.76	24.69	14.58	10
	2.47		3.59		5.37		4.98		7.14		3.54		30.79	
0.89	−0.02	2.12	0.60	3.63	2.57	3.07	1.94	3.48	1.24	2.57	1.33	24.77	18.90	7
	1.54		4.10		5.77		5.46		6.00		5.21		32.38	
0.64	−0.37	2.43	0.26	3.60	2.79	3.44	1.87	2.86	1.30	2.67	0.83	24.26	14.89	10
	2.88		3.47		6.87		4.18		4.98		4.56		30.46	
0.91	−0.45	2.53	0.41	3.63	1.92	3.10	1.91	2.97	1.15	2.61	0.50	24.77	16.29	10
	2.51		4.07		4.04		6.42		9.05		3.46		36.10	
0.95	−0.41	2.52	0.79	3.09	2.19	3.40	1.51	4.82	2.17	2.07	1.22	26.50	19.88	5
	1.70		4.91		6.85		8.10		4.99		4.81		31.71	
0.91	0.20	2.39	−0.23	3.63	0.81	3.75	2.44	2.47	0.72	2.53	0.84	24.99	15.91	9
	2.55		4.95		4.75		6.30		6.27		1.99		29.38	
1.47	0.24	2.42	0 65	3.16	1.78	3.47	1.87	3.35	1.58	1.75	1.21	25.14	16.00	5

TABLE 4.—YEARLY RAINFALL, RUN-OFF, RUN-OFF IN PERCENTAGE OF RAINFALL, AND LOSS.

SUSQUEHANNA BASIN.

Period.	Precipitation.			Run-off, in Inches.			Run-off, in Percentage of Precipitation.			Total Loss, in Inches.		
	Harrisburg, 24 030 sq. miles.	Wilkes-Barre, 9 810 sq. miles.	Williamsport, 5 640 sq. miles.	Harrisburg.	Wilkes-Barre.	Williamsport.	Harrisburg.	Wilkes-Barre.	Williamsport.	Harrisburg.	Wilkes-Barre.	Williamsport.
1891–92	45.17	.....	.....	24.90	.....	.....	55	....	....	20.27	.....	.....
1892–93	39.57	.....	.....	20.85	.....	.....	53	....	....	18.72	.....	.....
1893–94	39.01	.....	.....	21.06	.....	.....	54	....	....	17.95	.....	.....
1894–95	33.74	.....	.....	20.20	.....	.....	60	....	....	13.54	.....	.....
1895–96	37.57	.....	37.46	16.53	.....	16.57	44	....	42	21.04	.....	20.89
1896–97	36.66	.....	39.79	19.04	.....	21.74	52	....	55	17.62	.....	18.05
1897–98	39.77	.....	40.27	21.26	.....	23.00	53	....	57	18.51	.....	17.27
1898–99	39.75	.....	41.15	20.02	.....	20.69	50	....	50	19.73	.....	20.46
1899–00	31.62	31.77	33.04	16.34	15.15	17.34	52	47	52	15.28	16.62	15.70
1900–01	42.47	42.89	44.11	22.22	24.28	24.18	51	57	55	20.25	18.61	19.93
1901–02	43.75	44.13	43.79	25.71	27.09	27.58	59	61	63	18.04	17.04	16.21
1902–03	44.87	41.70	43.73	28.03	27.18	27.60	63	65	63	16.84	14.52	16.13
1903–04	40.84	41.91	42.51	21.07	27.13	25.86	52	65	61	19.77	14.78	16.65
1904–05	36.49	36.73	34.30	18.26	18.35	18.01	50	50	53	18.23	18.38	16.29
Mean	39.38	39.85	40.02	21.09	23.19	22.26	55	58	56	18.29	16.66	17.76
Maximum	45.17	44.13	44.11	28.03	27.18	27.60	63	65	63	21.04	18.61	20.89
Minimum	31.62	31.77	33.04	16.34	15.15	16.57	44	47	42	13.54	14.52	15.70

POTOMAC BASIN.

Period.	Precipitation.		Run-off, in Inches.		Run-off, in Percentage of Precipitation.		Total Loss, in Inches.	
	Point of Rocks, 9 650 sq. miles.	Millville, 3 000 sq. miles.	Point of Rocks.	Millville.	Point of Rocks.	Millville.	Point of Rocks.	Millville.
1895–96	37.25	42.10	8.16	9.05	22	21	29.09	33.05
1896–97	32.60	31.67	16.53	16.03	51	51	16.07	15.68
1897–98	38.50	40.83	13.47	12.14	35	30	25.03	28.64
1898–99	41.72	43.04	18.38	14.84	44	34	23.32	28.20
1899–00	30.09	33.63	8.99	9.73	30	29	21.10	23.90
1900–01	43.63	48.08	17.22	19.44	39	40	26.40	28.64
1901–02	35.33	34.36	21.46	19.73	61	58	13.87	14.58
1902–03	44.81	44.91	19.36	19.60	43	44	25.45	25.32
1903–04	29.37	30.47	9.30	7.86	32	23	20.07	22.61
1904–05	35.28	34.20	9.34	8.01	26	23	25.94	26.19
Mean	36.86	38.33	14.22	13.64	39	36	22.64	24.69
Maximum	44.81	48.08	21.46	19.78	61	58	29.09	33.05
Minimum	29.37	30.47	8.16	7.86	22	21	13.87	14.58

TABLE 4—(*Continued*).

JAMES BASIN.

Period.	Precipitation.				Run-off, in Inches.				Run-off, in Percentage of Precipitation.				Total Loss, in Inches.			
	Cartersville, 6 230 sq. miles.	Buchanan, 2 060 sq. miles.	Glasgow, 830 sq. miles.	Mattoax, 745 sq. miles.	Cartersville.	Buchanan.	Glasgow.	Mattoax.	Cartersville.	Buchanan.	Glasgow.	Mattoax.	Cartersville.	Buchanan.	Glasgow.	Mattoax.
1895–96	.....	44.12	45.22	.....	.....	12.20	14.76	.....	....	28	33	....	.....	31.92	30.46	....
1896–97	.....	34.25	32.92	.....	.....	17.10	16.63	.....	....	50	50	....	.....	17.15	16.29	....
1897–98	.....	42.69	38.78	.....	.....	12.78	12.81	.....	....	30	33	....	.....	29.91	25.97	....
1898–99	50.22	47.34	47.95	.....	22.20	21.28	17.35	.....	44	45	33	....	28.03	26.06	30.60	....
1899–00	36.57	35.78	36.49	.....	13.60	12.04	12.15	.....	37	34	42	....	22.97	23.74	24.34	....
1900–01	54.83	53.31	51.48	52.98	24.04	26.30	21.33	16.88	44	49	41	32	30.79	27.01	30.15	36.10
1901–02	37.83	35.72	35.54	38.13	18.93	20.83	18.81	17.51	50	58	53	46	18.90	14.89	16.73	20.62
1902–03	50.59	48.81	50.77	52.70	24.73	22.30	20.99	25.15	49	46	41	48	25.86	26.51	29.78	27.55
1903–04	30.58	30.45	32.48	30.80	10.69	11.45	12.78	10.92	35	38	39	35	19.89	19.00	19.70	19.88
1904–05	40.21	39.24	36.00	40.28	13.28	12.83	12.30	11.94	33	33	34	30	26.93	26.41	23.70	28.34
Mean	42.98	41.17	40.76	42.98	18.21	16.91	15.99	16.48	42	41	39	39	24.77	24.26	24.77	26.50
Maximum	54.83	53.31	51.48	52.98	24.73	26.30	21.33	25.15	50	58	53	48	30.79	32.38	30.46	36.10
Minimum	30.58	30.45	32.48	30.80	10.69	11.45	12.15	10.92	33	28	33	30	18.90	14.89	16.29	19.88

ROANOKE BASIN.

Period.	Precipitation.		Run-off, in Inches.		Run-off, in Percentage of Precipitation.		Total Loss, in Inches.	
	Randolph, 3 080 sq. miles.	Roanoke, 390 sq. miles.	Randolph.	Roanoke.	Randolph.	Roanoke.	Randolph.	Roanoke.
1896–97	.....	35.19	.....	14.90	.....	42	.....	20.29
1897–98	.....	38.28	.....	12.12	.....	32	.....	26.16
1898–99	.....	48.85	.....	27.32	.....	56	.....	21.53
1899–00	.....	38.08	.....	12.32	.....	32	.....	25.76
1900–01	53.95	58.30	25.16	29.66	47	51	28.79	28.64
1901–02	37.14	34.47	21.14	18.56	57	54	16.00	15.91
1902–03	50.99	49.73	22.50	21.15	44	43	28.49	28.58
1903–04	34.00	35.21	10.99	8.88	32	25	23.01	26.33
1904–05	42.89	46.03	13.51	14.32	32	31	29.38	31.71
Mean	43.80	42.68	18.66	17.69	43	41	25.14	24.99
Maximum	53.95	58.30	25.16	29.66	57	56	29.38	31.71
Minimum	34.00	34.47	10.99	8.88	32	25	16.00	15.91

TABLE 5.—YEARLY RAINFALL, RUN-OFF, RUN-

Period.	RAINFALL, IN INCHES.							RUN-OFF, IN INCHES.						
	Connecticut, above Orford, N. H. 3 300 sq. miles.	Housatonic, above Gaylordsville, Conn. 1 020 sq. miles.	Susquehanna, above Harrisburg, Pa. 24 030 sq. miles.	Ohio, above Wheeling, W. Va. 23 820 sq. miles.	Potomac, above Point of Rocks, Md. 9 650 sq. miles.	James, above Cartersville, Va. 6 230 sq. miles.	Roanoke, above Roanoke, Va. 390 sq. miles.	Connecticut, above Orford, N. H.	Housatonic, above Gaylordsville, Conn.	Susquehanna, above Harrisburg, Pa.	Ohio, above Wheeling, W. Va.	Potomac, above Point of Rocks, Md.	James, above Cartersville, Va.	Roanoke, above Roanoke, Va.
1884–85	.....	.....	.....	39.12	.....	.....	.....	.....	.....	.....	17.82	.....	.....	.....
1885–86	.....	.....	.....	39.89	.....	.....	.....	.....	.....	.....	21.18	.....	.....	.....
1886–87	.....	.....	.....	39.34	.....	.....	.....	.....	.....	.....	22.39	.....	.....	....
1887–88	.....	.....	.....	37.46	.....	.....	.....	.....	.....	.....	17.46	.....	.....	.....
1888–89	.....	.....	.....	39.52	.....	.....	.....	.....	.....	.....	24.34	.....	.....	.....
1889–90	.....	.....	.....	55.56	.....	.....	.....	.....	.....	.....	34.20	.....	.....	.....
1890–91	.....	.....	.....	48.63	.....	.....	.....	.....	.....	.....	30.64	.....	.....	.....
1891–92	.....	.....	45.17	43.40	.....	.....	.....	.....	.....	24.90	22.09	.....	.....	.....
1892–93	.....	.....	39.57	39.40	.....	.....	.....	.....	.....	20.85	21.52	.....	.....	.....
1893–94	.....	.....	39.01	38.55	.....	.....	.....	.....	.....	21.06	18.03	.....	.....	.....
1894–95	.....	.....	33.74	33.47	.....	.....	.....	.....	.....	20.20	16.29	.....	.....	.....
1895–96	.....	.....	37.57	44.57	37.25	.....	.....	.....	.....	16.53	19.71	8.16	.....	.....
1896–97	.....	.....	36.66	38.44	32.60	.....	35.19	.....	.....	19.04	22.59	16.53	.....	14.90
1897–98	.....	.....	39.77	46.51	38.50	.....	38.28	.....	.....	21.26	24.61	13.47	.....	12.12
1898–99	.....	.....	39.75	43.26	41.72	50.23	48.85	.....	.....	20.02	22.88	18.38	22.20	27.32
1899–00	.....	.....	31.62	35.94	30.09	36.57	38.08	.....	.....	16.34	17.64	8.99	13.60	12.32
1900–01	38.93	48.24	42.47	45.21	43.63	54.83	58.30	24.00	25.68	22.22	24.55	17.22	24.04	29.66
1901–02	41.80	51.49	43.75	40.78	35.33	37.83	34.47	27.04	33.17	25.71	24.66	21.46	18.93	18.56
1902–03	34.76	50.33	44.87	45.20	44.81	50.59	49.73	21.92	36.94	28.03	27.09	19.36	24.73	21.15
1903–04	34.82	49.48	40.84	41.62	29.37	30.58	35.21	16.01	27.61	21.07	25.35	9.30	10.69	8.88
1904–05	33.48	39.77	36.49	39 91	35.28	40.21	46.03	19.31	23.76	18.26	21.19	9.34	13.28	14.32
Mean	36.76	47.86	39.38	41.71	36.86	42.98	42.68	21.66	29.43	21.09	22.68	14.22	18.21	17.69
Maximum	41.80	51.49	45.17	55.56	44.81	54.83	58.30	27.04	36.94	28.03	34.20	21.46	24 73	29.66
Minimum	33.48	39.77	31.62	33.47	29.37	30.58	34 47	16.01	23.76	16.34	16.29	8.16	10.69	8.88

OFF IN PERCENTAGE OF RAINFALL, AND LOSS.

Run-off, in Percentage of Rainfall.							Total Loss, in Inches.							Period.
Connecticut, above Orford, N. H.	Housatonic, above Gaylordsville, Conn.	Susquehanna, above Harrisburg, Pa.	Ohio, above Wheeling, W. Va.	Potomac, above Point of Rocks, Md.	James, above Cartersville, Va.	Roanoke, above Roanoke, Va.	Connecticut, above Orford, N. H.	Housatonic, above Gaylordsville, Conn.	Susquehanna, above Harrisburg, Pa.	Ohio, above Wheeling, W. Va.	Potomac, above Point of Rocks, Md.	James, above Cartersville, Va.	Roanoke, above Roanoke, Va.	
.....	.....	.....	46	.....	.....	.....	.....	.....	.....	21.30	.....	.....	.....	1884-85
.....	.....	.....	53	.....	.....	.....	.....	.....	.....	18.71	.....	.....	.....	1885-86
.....	.....	.....	57	.....	.....	.....	.....	.....	.....	16.95	.....	.....	.....	1886-87
.....	.....	.....	47	.....	.....	.....	.....	.....	.....	20.00	.....	.....	.....	1887-88
.....	.....	.....	62	.....	.....	.....	.....	.....	.....	15.18	.....	.....	.....	1888-89
.....	.....	.....	62	.....	.....	.....	.....	.....	.....	21.36	.....	.....	.....	1889-90
.....	.....	.....	63	.....	.....	.....	.....	.....	.....	17.99	.....	.....	.....	1890-91
.....	.....	55	51	.....	.....	.....	.....	.....	20.27	21.31	.....	.....	.....	1891-92
.....	.....	53	55	.....	.....	.....	.....	.....	18.72	17.88	.....	.....	.....	1892-93
.....	.....	54	47	.....	.....	.....	.....	.....	17.95	20.52	.....	.....	.....	1893-94
.....	.....	60	49	.....	.....	.....	.....	.....	13.54	17.18	.....	.....	.....	1894-95
.....	.....	44	44	22	.....	.....	.....	.....	21.04	24.86	29.09	.....	.....	1895-96
.....	.....	52	59	51	.....	42	.....	.....	17.62	15.85	16.07	.....	20.29	1896-97
.....	.....	53	53	35	.....	32	.....	.....	18.51	21.90	25.03	.....	26.16	1897-98
.....	.....	50	53	44	44	56	.....	.....	19.73	20.38	23.32	28.03	21.53	1898-99
.....	.....	52	49	30	37	32	.....	.....	15.38	18.30	21.10	22.97	25.76	1899-00
62	53	51	54	39	44	51	14.93	22.56	20.25	20.48	26.40	30.79	28.64	1900-01
65	64	59	60	61	50	54	14.76	18.32	18.04	16.12	13.87	18.90	15.91	1901-02
63	73	63	60	43	49	43	12.84	13.39	16.84	18.11	25.45	25.86	28.58	1902-03
46	56	52	61	32	35	25	18.81	21.87	19.77	16.27	20.07	19.89	26.38	1903-04
58	60	50	53	26	33	31	14.17	16.01	18.23	18.72	25.94	26.93	31.71	1904-05
59	62	55	54	39	42	41	15.10	18.43	18.29	19.02	22.64	24.77	24.99	Mean.
65	73	62	63	61	50	56	18.81	22.56	21.04	24.86	29.09	30.79	31.71	Maximum.
46	53	44	44	22	33	25	12.84	13.39	13.54	15.18	13.87	18.90	15.91	Minimum.

TABLE 6.—RAINFALL, RUN-OFF, RUN-OFF IN PERCENTAGE OF RAINFALL, AND LOSS, FOR THE WINTER AND THE SUMMER MONTHS.

CONNECTICUT, AT ORFORD, N. H.

Year.	Winter Months. Dec. to Apr., Inclusive.				Summer Months, June, July, August.				Loss for year.
	Rainfall.	Run-off.	Percentage of run-off.	Loss.	Rainfall.	Run-off.	Percentage of run-off.	Loss.	
1900-1901	11.43	12.10	106	—0.67	11.07	4.21	38	6.86	14.93
1901-1902	16.99	14.78	87	2.21	12.59	6.22	49	6.37	14.76
1902-1903	15.53	14.60	94	0.93	12.49	2.59	21	9.90	12.84
1903-1904	11.60	6.81	59	4.79	10.17	2.20	22	7.97	18.81
1904-1905	8.87	7.66	86	1.21	13.65	4.12	30	9.53	14.17
Mean	12.89	11.19	87	1.70	12.00	3.87	32	8.13	15.10
Maximum	16.99	14.78	106	4.79	13.65	6.22	49	9.90	18.81
Minimum	8.87	6.81	59	—0.67	10.17	2.20	21	6.37	12.84

HOUSATONIC, ABOVE GAYLORDSVILLE, CONN.

1900-1901	16.45	14.20	86	2.25	14.15	4.44	31	9.71	22.56
1901-1902	20.91	21.63	103	—0.72	16.11	5.04	31	11.07	18.32
1902-1903	20.23	21.40	106	—1.17	21.48	8.96	42	12.52	13.39
1903-1904	16.92	13.61	80	3.31	13.93	4.83	35	9.10	21.87
1904-1905	14.48	14.74	102	—0.26	14.43	2.35	16	12.08	16.01
Mean	17.80	17.12	96	0.68	16.02	5.12	32	10.90	18.43
Maximum	20.91	21.63	106	3.31	21.48	8.96	42	12.52	22.56
Minimum	14.48	13.61	80	—1.17	13.93	2.35	16	9.10	13.39

SUSQUEHANNA, ABOVE HARRISBURG, PA.

1891-1892	15.85	13.97	88	1.88	14.93	4.71	32	10.22	20.27
1892-1893	14.87	13.20	89	1.67	11.25	1.62	14	9.63	18.72
1893-1894	13.49	11.00	82	2.49	7.42	3.11	42	4.31	17.95
1894-1895	11.99	14.17	118	—2.18	10.06	1.20	12	8.86	13.54
1895-1896	15.29	12.97	85	2.32	11.40	2.32	20	9.08	21.04
1896-1897	11.39	10.76	94	0.63	10.88	2.09	19	8.79	17.62
1897-1898	15.14	14.06	93	1.08	12.24	2.56	21	9.68	18.51
1898-1899	13.51	13.75	102	—0.24	10.09	1.26	12	8.83	19.73
1899-1900	13.96	12.99	93	0.97	9.67	1.19	12	8.48	15.28
1900-1901	12.84	12 02	94	0.82	13.16	4.81	37	8.35	20.25
1901-1902	18.10	17.97	99	0.13	16.17	5.14	32	11.03	18.04
1902-1903	18.84	19.15	102	—0.31	17.44	4.09	23	13.35	16.84
1903-1904	14.60	12.50	86	2.10	12.26	2.72	22	9.54	19.77
1904-1905	12.82	11.82	92	1.00	14.40	3.13	22	11.27	18.28
Mean	14.48	13.58	94	0.90	12.25	2.85	23	9.40	18.39
Maximum	18.84	19.15	118	2.49	17.44	5.14	42	13.35	21.04
Minimum	11.39	10.76	82	—2.18	7.42	1.19	12	4.31	13.54

TABLE 6—*(Continued)*.

SUSQUEHANNA, ABOVE WILKES-BARRE, PA.

Year.	Winter Months, Dec. to Apr., Inclusive.				Summer Months, June, July, August.				Loss for year.
	Rainfall.	Run-off.	Percentage of run-off.	Loss.	Rainfall.	Run-off.	Percentage of run-off.	Loss.	
1899–1900	14.17	12.42	88	1.75	9.80	0.88	9	8.92	16.62
1900–1901	13.18	16.90	128	—3.72	13.10	3.01	23	10.09	18.61
1901–1902	16.66	20.85	125	—4.19	16.61	4.10	25	12.51	17.04
1902–1903	16.68	19.20	115	—2.52	17.28	3.62	21	13.66	14.52
1903–1904	13.79	17.80	129	—4.01	12.49	2.31	18	10.18	14.78
1904–1905	12.32	11.72	95	0.60	14.77	2.49	17	12.28	18.38
Mean	14.47	16.48	114	—2.01	14.00	2.74	20	11.26	16.66
Maximum	16.68	20.85	129	1.75	17.28	4.10	25	13.66	18.61
Minimum	12.32	11.72	88	—4.19	9.80	0.88	9	8.92	14.52

SUSQUEHANNA, AT WILLIAMSPORT, PA.

1895–1896	14.53	10.97	76	3.56	12.49	4.06	33	8.43	20.89
1896–1897	13.22	12.01	91	1.21	11.76	2.06	18	9.70	18.05
1897–1898	16.95	16.78	101	0.17	11.76	2.18	19	9.58	17.27
1898–1899	14.50	14.44	100	0.06	10.55	1.17	11	9.38	20.46
1899–1900	15.32	13.31	87	2.01	9.81	1.49	15	8.32	15.70
1900–1901	13.27	13.10	99	0.17	14.10	4.48	32	9.62	19.93
1901–1902	18.41	19.23	104	—0.82	16.02	5.78	36	10.24	16.21
1902–1903	18.53	19.84	107	—1.31	16.77	4.79	29	11.98	16.13
1903–1904	17.57	16.71	95	0.86	11.75	3.01	26	8.74	16.65
1904–1905	12.52	11.25	90	1.27	13.68	4.00	29	9.68	16.29
Mean	15.48	14.76	95	0.72	12.87	3.30	26	9.57	17.76
Maximum	18.53	19.84	107	3.56	16.77	5.78	33	11.98	20.89
Minimum	12.52	10.97	76	—1.31	9.81	1.17	11	8.32	15.70

TABLE 6—(*Continued*).

OHIO, ABOVE WHEELING, W. VA.

Year.	Winter Months, Dec. to Apr., Inclusive.				Summer Months, June, July, August.				Loss for year.
	Rainfall.	Run-off.	Percentage of run-off.	Loss.	Rainfall.	Run-off.	Percentage of run-off.	Loss.	
1884-1885	14.09	11.80	84	2.29	14.66	3.12	21	11.54	21.30
1885-1886	14.30	15.11	106	—0.81	10.84	2.00	18	8.84	18.71
1886-1887	16.71	16.19	97	0.52	10.73	1.68	16	9.05	16.95
1887-1888	13.46	10.95	81	2.51	12.58	3.79	30	8.79	20.00
1888-1889	12.90	12.12	94	0.78	11.59	5.44	47	6.15	15.18
1889-1890	22.03	20.51	93	1.52	12.39	2.66	21	9.73	21.36
1890-1891	18.70	18.95	101	—0.25	15.29	3.99	26	11.30	17.99
1891-1892	15.99	13.50	84	2.49	12.70	3.75	30	8.95	21.31
1892-1893	17.87	14.96	84	2.91	9.83	1.20	12	8.63	17.88
1893-1894	15.75	11.99	76	3.76	6.34	1.66	26	4.68	20.52
1894-1895	14.25	13.23	93	1.02	9.91	1.04	10	8.87	17.18
1895-1896	14.93	12.27	82	2.66	17.14	5.57	33	11.57	24.86
1896-1897	14.93	14.61	98	0.32	12.87	2.63	20	10.24	15.85
1897-1898	19.34	17.80	92	1.54	14.46	3.10	21	11.36	21.90
1898-1899	15.39	15.48	99	—0.09	11.01	2.06	19	8.95	20.38
1899-1900	15.24	13.94	91	1.30	12.57	1.93	15	10.64	18.30
1900-1901	15.77	15.99	101	—0.22	12.57	3.35	27	9.22	20.48
1901-1902	16.39	17.07	104	—0.68	14.56	5.12	35	9.44	16.12
1902-1903	20.73	20.79	100	—0.06	15.09	3.54	23	11.55	18.11
1903-1904	17.46	17.43	100	0.03	11.99	3.43	29	8.56	16.27
1904-1905	14.56	13.81	95	0.75	15.75	4.39	28	11.36	18.72
Mean	16.23	15.16	93	1.07	12.61	3.12	25	9.49	19.02
Maximum	22.03	20.51	106	3.76	17.14	5.57	47	11.57	24.86
Minimum	12.90	10.95	76	—0.81	6.34	1.04	10	6.15	15.18

POTOMAC, ABOVE POINT OF ROCKS, MD.

1895-1896	13.20	5.16	39	8.04	12.49	2.15	17	10.34	29.09
1896-1897	12.85	9.41	73	3.44	9.44	1.82	19	7.62	16.07
1897-1898	14.87	7.11	48	7.76	12.02	3.43	29	8.59	25.03
1898-1899	15.22	12.92	85	2.30	8.59	1.19	14	7.40	23.32
1899-1900	12.24	6.17	50	6.07	10.01	1.56	16	8.45	21.10
1900-1901	13.69	7.97	58	5.72	14.61	4.46	31	10.15	26.40
1901-1902	19.92	18.50	93	1.42	8.36	1.03	12	7.33	13.87
1902-1903	18.84	13.24	70	5.60	14.91	4.18	28	10.73	25.45
1903-1904	8.57	5.37	63	3.20	11.64	2.00	17	9.64	20.07
1904-1905	11.98	5.46	46	6.52	15.94	2.68	17	13.26	25.94
Mean	14.14	9.14	65	5.00	11.80	2.44	21	9.36	22.64
Maximum	19.92	18.50	93	8.04	15.94	4.46	31	13.26	29.09
Minimum	8.57	5.16	39	1.42	8.36	1.03	12	7.33	13.87

TABLE 6—(*Continued*).

SHENANDOAH, ABOVE MILLVILLE, W. VA.

Year.	Winter Months, Dec. to Apr., Inclusive.				Summer Months, June, July, August.				Loss for year.
	Rainfall.	Run-off.	Percentage of run-off.	Loss.	Rainfall.	Run-off.	Percentage of run-off.	Loss.	
1895–1896	14.85	5.10	34	9.75	15.12	2.30	15	12.82	33.05
1896–1897	12.16	8.17	67	3.99	8.73	1.54	18	7.19	15.68
1897–1898	13.70	4.60	34	9.10	14.77	4.24	29	10.53	28.64
1898–1899	16.88	8.93	53	7.95	8.54	1.32	15	7.22	28.20
1899–1900	13.25	6.31	48	6.94	11.67	1.69	14	9.98	23.90
1900–1901	14.51	8.11	56	6.40	17.84	6.14	34	11.70	28.64
1901–1902	19.11	15.98	84	3.13	7.32	1.35	18	5.97	14.58
1902–1903	18.72	12.31	66	6.41	14.22	4.72	33	9.50	25.32
1903–1904	8.74	3.58	41	5.16	13.14	2.06	16	11.08	22.61
1904–1905	11.80	4.08	35	7.72	14.67	2.70	18	11.97	26.19
Mean	14.38	7.72	54	6.66	12.60	2.80	22	9.80	24.69
Maximum	19.11	15.98	84	9.75	17.84	6.14	34	12.82	33.05
Minimum	8.74	3.58	34	3.13	8.54	1.32	14	5.97	14.58

JAMES, ABOVE CARTERSVILLE, VA.

1898–1899	20.45	15.07	74	5.38	11.83	1.66	14	10.15	28.03
1899–1900	16.26	8.94	55	7.32	9.86	2.27	23	7.59	22.97
1900–1901	16.44	9.78	60	6.66	21.07	7.58	36	13.49	30.79
1901–1902	20.54	14.70	72	5.84	8.98	1.66	18	7.32	18.90
1902–1903	21.63	15.75	73	5.88	15.17	4.48	30	10.69	25.86
1903–1904	10.43	5.32	51	5.11	11.33	2.32	20	9.01	19.89
1904–1905	12.89	5.78	45	7.11	17.55	4.57	26	12.98	26.93
Mean	16.95	10.76	63	6.19	13.69	3.51	26	10.18	24.77
Maximum	21.63	15.75	74	7.32	21.07	7.58	36	13.49	30.79
Minimum	10.43	5.32	45	5.11	8.98	1.66	14	7.32	18.90

JAMES, ABOVE BUCHANAN, VA.

1895–1896	16.85	7.17	43	9.68	15.31	2.91	19	12.40	31.92
1896–1897	12.89	10.65	83	2.24	9.29	1.49	16	7.80	17.15
1897–1898	14.47	6.32	44	8.15	14.14	3.21	23	10.93	29.91
1898–1899	18.14	14.50	80	3.64	10.61	1.10	10	9.51	26.06
1899–1900	15.36	8.72	57	6.64	10.54	1.90	18	8.64	23.74
1900–1901	15.94	11.12	70	4.81	19.55	7.32	37	12.23	27.01
1901–1902	20.72	17.99	87	2.73	7.57	1.21	16	6.36	14.89
1902–1903	22.29	16.02	72	6.27	14.04	3.47	25	10.57	26.51
1903–1904	10.34	5.86	57	4.48	11.69	2.52	22	9.17	19.00
1904–1905	12.85	5.38	42	7.47	16.00	4.51	28	11.49	26.41
Mean	15.99	10.37	65	5.62	12.87	2.97	23	9.90	24.26
Maximum	22.29	17.99	87	10.14	19.55	7.32	37	12.40	31.92
Minimum	10.34	5.38	43	2.24	7.57	1.10	10	6.36	14.89

TABLE 6—(*Continued*).

NORTH (OF JAMES) ABOVE GLASGOW, VA.

Year.	Winter Months, Dec. to Apr., Inclusive.				Summer Months, June, July, August.				Loss for year.
	Rainfall.	Run-off.	Percentage of run-off.	Loss.	Rainfall.	Run-off.	Percentage of run-off.	Loss.	
1895–1896	16.94	7.67	45	9.27	15.68	3.43	22	12.95	30.46
1896–1897	11.88	9.24	78	2.64	9.58	1.44	15	8.14	16.29
1897–1898	12.94	5.52	43	7.42	12.67	3.90	31	8.77	25.97
1898–1899	18.90	11.74	62	7.16	10.61	0.82	8	9.79	30.60
1899–1900	15.28	8.26	54	7.02	10.21	2.15	21	8.06	24.34
1900–1901	16.33	9.33	57	7.00	18.55	6.67	36	11.88	30.15
1901–1902	20.31	16.14	79	4.17	6.93	0.98	14	5.95	16.73
1902–1903	21.82	14.78	68	7.04	16.76	3.46	21	13.30	29.78
1903–1904	11.43	7.55	66	3.88	11.28	2.38	21	8.90	19.70
1904–1905	13.06	5.45	42	7.61	14.60	4.54	31	10.06	23.70
Mean	15.89	9.57	60	6.32	12.69	2.98	23	9.71	24.77
Maximum	21.82	16.14	79	9.27	18.55	6.67	36	13.30	30.46
Minimum	11.43	5.45	42	2.64	6.93	0.82	8	5.95	16.29

APPOMATTOX, ABOVE MATTOAX, VA.

1900–1901	16.70	6.74	40	9.96	19.93	5.61	28	14.32	36.10
1901–1902	21.83	13.04	60	8.79	10.54	2.11	20	8.43	20.62
1902–1903	22.86	16.55	72	6.31	13.99	3.90	28	10.09	27.55
1903–1904	9.91	5.90	60	4.01	10.74	1.72	16	9.02	19.88
1904–1905	13.03	7.24	56	5.79	16.60	1.90	11	14.70	28.34
Mean	16.87	9.89	59	6.98	14.36	3.05	21	11.31	26.50
Maximum	22.86	16.55	72	9.96	19.93	5.61	28	14.70	36.10
Minimum	9.91	5.90	40	4.01	10.54	1.72	11	8.43	19.88

ROANOKE, ABOVE ROANOKE, VA.

1896–1897	14.65	8.26	56	6.39	9.45	2.16	23	7.29	20.29
1897–1898	12.67	4.74	37	7.93	10.73	3.08	29	7.65	26.16
1898–1899	21.47	19.80	92	1.67	11.91	1.50	13	10.41	21.53
1899–1900	15.73	8.27	53	7.46	10.43	2.12	20	8.31	25.76
1900–1901	16.14	10.34	64	5.80	23.04	10.82	47	12.22	28.64
1901–1902	20.26	15.03	74	5.23	8.06	1.23	15	6.83	15.91
1902–1903	22.31	14.50	65	7.81	12.97	2.87	22	10.10	28.58
1903–1904	9.95	3.23	32	6.72	16.55	3.32	20	13.23	26.33
1904–1905	15.29	4.43	29	10.86	18.22	5.56	30	12.66	31.71
Mean	16.50	9.84	60	6.66	13.48	3.63	27	9.85	24.99
Maximum	22.31	19.80	92	10.86	23.04	10.82	47	13.23	31.71
Minimum	9.95	3.23	29	1.67	8.06	1.23	13	6.83	15.91

TABLE 6—(*Continued*).

ROANOKE, ABOVE RANDOLPH, VA.

Station.	Winter Months, Dec. to Apr., Inclusive.				Summer Months, June, July, August.				Total yearly loss.
	Rainfall.	Run-off.	Percentage of run-off.	Loss.	Rainfall.	Run-off.	Percentage of run-off.	Loss.	
1900–1901	16.56	9.50	57	7.06	20.81	9.01	43	11.80	28.79
1901–1902	21.26	13.95	66	7.31	9.06	2.96	33	6.10	16.00
1902–1903	24.65	13.88	56	10.77	12.91	3.43	27	9.48	28.49
1903–1904	10.17	4.64	46	5.53	14.47	3.11	22	11.36	23.01
1904–1905	14.94	5.61	38	9.33	15.79	4.53	29	11.16	29.38
Mean	17.53	9.53	54	8.00	14.60	4.62	32	9.98	25.14
Maximum	24.65	13.95	66	10.77	20.81	9.01	43	11.80	29.98
Minimum	10.17	4.64	38	5.53	9.06	2.96	22	6.10	16.00

FOR THE MEAN YEAR.

Connecticut, at Orford, N. H.	12.89	11.19	87	1.76	12.00	3.87	32	8.13	15.10
Housatonic, at Gaylordsville, Conn.	17.80	17.12	96	0.68	16.02	5.12	32	10.90	18.43
Susquehanna, at Harrisburg, Pa.	14.48	13.58	94	0.90	12.25	2.85	23	9.40	18.29
Susquehanna, at Wilkes-Barre, Pa.	14.47	16.48	114	—2.01	14.00	2.74	20	11.26	16.66
Susquehanna, at Williamsport, Pa.	15.48	14.76	95	0.72	12.87	3.30	26	9.57	17.76
Ohio, at Wheeling, W. Va.	13.23	15.16	93	1.07	12.61	3.12	25	9.49	19.02
Potomac, at Point of Rocks, Md.	14.14	9.14	65	5.00	11.80	2.44	21	9.36	22.64
Shenandoah, at Millville, W. Va.	14.38	7.72	54	6.66	12.60	2.80	22	9.80	24.69
James, at Cartersville, Va.	16.95	10.76	63	6.19	13.69	3.51	26	10.18	24.77
James, at Buchanan, Va.	15.99	10.37	65	5.62	12.87	2.97	23	9.99	24.26
North (of James) Glasgow, Va.	15.89	9.57	60	6.32	12.69	2.98	23	9.71	24.77
Appomattox, at Mattoax, Va.	16.87	9.89	59	6.98	14.36	3.05	21	11.31	26.50
Roanoke, at Roanoke, Va.	16.50	9.84	60	6.66	13.48	3.63	27	9.85	24.99
Roanoke, at Randolph, Va.	17.53	9.53	54	8.00	14.60	4.62	32	9.98	25.14

DISCUSSION.

Mr. Murphy. E. C. MURPHY, ASSOC. M. AM. SOC. C. E. (by letter).—The writer believes that the data available for computing run-off from precipitation will not justify the use of this method for estimating stream flow, even in the eastern part of the United States, where the number of precipitation stations per unit area is largest and the records longest. His opinion is based on measurements of run-off and a study of the corresponding precipitation data. He has observed large floods on streams, that the precipitation data do not indicate, because the stations were too few; he has also observed annual run-off larger than the observed annual precipitation, due to the stations not being properly located with respect to altitude. He believes that writers on this subject do not, as a rule, treat it fairly. While they admit that the available data are unsatisfactory at best—on account of lack of funds for its collection, and lack of data for computing the effect of some of the quantities upon which run-off depends—nevertheless, they draw conclusions from their results as if the data were good and complete, and intimate that the results obtained by this method are as good, if not better, than those obtained by gauging. They subject their data, such as they are, to various mathematical processes which have little, if any, experimental basis, and in some mysterious way much value is imparted to them. To illustrate: a recent user of this method, when he found the measured monthly precipitation at any station more than twice the mean for that month, substituted the mean for the observed quantity. That is, when perchance one of the few precipitation stations was near the center of a storm area, and its record was approximately the rainfall corresponding to the measured run-off, he threw away the record and used in its place a quantity less than half the observed quantity. By this and similar processes he finally obtained what he called "deduced mean annual dependable precipitation." His computed run-off is so reliable that "stream gaugings were only used as a guide."

Run-off is a resultant quantity; its value depends upon several quantities or factors. It can be obtained directly by measurement of the flow of the streams, or it can be computed from the measurements and proper combination of the quantities upon which it depends. Precipitation, while the most important, is only one of these factors; temperature, wind velocity, barometric pressure, surface and ground storage, shape and extent, inclination and character of water-shed, extent of forest and cultivated area, are other factors. This is the formidable array of quantities which the user of this method must properly reckon with in order to get a reasonably accurate estimate of run-off.

Mr. Murphy.

Let us see with what accuracy precipitation on a drainage basin is measured. It is admitted by all that precipitation is an erratic phenomenon; it does not follow any known law; it cannot be obtained by any mathematical process, and the only way to obtain its accurate value is by measurement or observation. The monthly precipitation at two stations only a few miles apart may differ more than 25 per cent. It varies in irregular cycles, but this knowledge does not help in the least in supplying missing data. It is seen to depend on elevation, topography and direction of the prevailing wind, but there are not sufficient data to enable the effect of these conditions to be computed with reasonable accuracy.

A measurement of rainfall is subject to two errors from air-currents, which may amount to 25% of the catch. Snow measurements are subject to larger error than rainfall measurements. A rain-gauge has a diameter of only 8 in. and shows the rain falling on an area of less than 0.35 sq. ft. What confidence can be placed in the precipitation on a water-shed computed from available data? Take, for example, the Susquehanna River water-shed, above Williamsport, Pa., mentioned by the author. There are eight precipitation stations in an area of 5 640 sq. miles—one station to 725 sq. miles. The precipitation on 725 sq. miles is computed from that on less than half a square foot. This station is located of necessity at a convenient place, each measurement is subject to an error of 25% of the catch, and the law of change between stations is not known. The engineer has very little confidence in such data, and will only use it for rough preliminary estimates.

Wind movement is neglected entirely. From evaporation measurements it is known that the loss from a body of water increases as rapidly as the wind velocity increases. How is it possible to get such reliable estimates of run-off by this method, when so important a quantity as wind movement is neglected?

In striking contrast with all this uncertainty from lack of data is the simple direct method of obtaining run-off by gauging the streams. This method gives the run-off for each month and day, as well as the annual run-off. If a meteorological station were maintained in each township (36 sq. miles) where not only precipitation, temperature, wind velocity, and atmospheric pressure, but all the other quantities upon which run-off depends, were measured, data would then be available for a reasonably accurate estimate of run-off from rainfall. It would be folly, however, to spend so large a sum for run-off data, when the run-off can be measured for a much less sum.

In the early days of the settlement of the United States, there was a great demand for meteorological data, mainly for agricultural purposes, and no demand for run-off data; at the present time, there is a great demand in many parts of the country for run-off data. Stream gauging alone can satisfy this demand.

Mr. Murphy. The writer does not wish to convey the idea that the method of computing run-off from precipitation should never be used. It may be used with profit in some sections of the country in connection with stream gauging. If, for example, the flow of a stream at a given place is known from stream gaugings for, say, three years, and the precipitation over the basin is also known, with a reasonable accuracy, for these same three years and, say, the preceding five years, then an approximate estimate of the annual run-off for the latter period may be computed from the precipitation of that period, together with a comparison of precipitation and run-off for the former period when the gaugings were made. In other words, this method may be used to supplement stream-gauging records, but, to compute run-off from meteorological data without gaugings, appears, to the writer, to be a waste of time.

The writer would prefer not to discuss Mr. Merriman's criticism of the run-off data of the Water Resources Branch of the United States Geological Survey, were it not that some of those who read his discussion may form a wrong opinion of the accuracy of this work. The writer has been a member of this branch of the United States Geological Survey for twelve years, and has made numerous tests of the accuracy of the data obtained by it. During his three years of service as Chief Inspector of Stream Gauging work, and in charge of the flood investigations, he has visited almost all the important gauging stations in the United States, has had an excellent opportunity to judge of the accuracy of the work in question, and does not hesitate to take issue with Mr. Merriman's criticism.

The officials in charge of this branch have always encouraged the hydrographers in their efforts to improve the instruments and methods, and have instructed them to test, as far as possible, their results and the observers' gauged records. A large number of data have been thus obtained and published, bearing on the degree of accuracy of the results obtained. The writer has no doubt that a study of the methods used and of the results of these tests will convince any engineer that the results are not only not uniformly 10% too large, but that they are remarkably accurate, taking into account the small amount of funds available, the large number of data obtained, and the difficulties in the way of securing accurate data in some parts of the country.

In this matter it may be permissible to refer to studies of the degree of accuracy of current-meter work made by the writer for this branch. In 1900 he was instructed to make a thorough study of the methods of stream gauging in use, with a view to their improvement, and the determination of the degree of accuracy of results obtained by them. This study was made at the Cornell University Hydraulic Laboratory. Discharges varying from 10 to 200 cu. ft. per sec. were measured with a standard weir, and with all the types of current

meters then in use by this branch, operated by all the different methods in use for measuring discharge. The meters were rated at the United States Geological Survey rating stations, and the ratings were tested at the laboratory, using a chronograph for recording the time and revolutions. In these experiments the meters were held with a rod, as well as by a cable, wherever possible. Mr. Murphy.

In the case of the Price meter: When held with a rod, the meter was free to rotate about a horizontal axis; in other cases it was not free to rotate about a horizontal axis. While being rated, the meter was held in the same manner as when measuring discharge. Some of the experiments were made in a canal 16 ft. wide, the remainder in a canal about 4 ft. wide. In short, in every way known to the writer, the instruments, methods, and results were tested by comparison with corresponding quantities simultaneously obtained by a standard weir. At the close of this laboratory work, this study was extended to streams.

In 1903 he made a study in this laboratory of the accuracy of the low-water measurements of flow of western streams, namely, small, shallow streams flowing over a gravel or cobble bed.

It is not necessary to give here the results of these studies. They are all given in United States Geological Survey Water Supply Paper No. 96, some of them in the *Transactions** of the American Society of Civil Engineers.

If Mr. Merriman will study the results of these tests and those of R. E. Horton, H. K. Barrows, F. W. Hanna, and other members of this service, he will find that he has formed his opinion too hastily, and that experience in the measurement of stream discharge under a variety of channel conditions shows that the small Price meter, held with a cable, is quite well suited to his use; that the error in velocity due to the meter being held with a cable is not more than half of 10% at good gauging stations, for velocities of more than 2 ft. per sec.; that the error in run-off, due to the current meter, is greatest at low velocities, makes the discharge too small, instead of too large, and that it decreases as the velocity increases, as long as the meter can be held in its proper place. He will find that the chief source of error in stream-gauging work is not due to the current meter, but to changing channel conditions and rapid change of stage. Also, that at the good gauging stations of the United States Geological Survey, although some of the individual discharge measurements, when plotted, may fall 5% on either side of the discharge curve, the discharge taken from the discharge curve is probably not in error more than 3%, except for very low velocities, when it may in some cases be 6 or even 10% too small, and for very high stages, when the stream overflows its banks. In the latter case the discharge is just as likely to be too small as too large.

* Vol. XLVII, p. 370.

Mr. Murphy. Even if the run-off from two near-by water-sheds, in which the physical characteristics are the same to all outward appearance, may differ 25%, and if the run-off from two other water-sheds differs by less than half that amount, how can Mr. Merriman, by any application of meteorological data, correctly infer that the run-off from one of the latter is in error 10 per cent.? The writer believes that Mr. Merriman has allowed his faith in run-off computed from meteorological data to lead him to erroneous conclusions.

Mr. Holden. CHARLES A. HOLDEN, ASSOC. AM. SOC. C. E. (by letter).—The writer is in hearty accord with the efforts of Mr. Hoyt to bring about the estimation of available water supply through the use of run-off data instead of the comparatively uncertain precipitation data.

Precipitation data must always give a very imperfect record, because, of necessity, observation stations are situated only at infrequent intervals and at varying positions in elevation. All are familiar with the phenomenon of heavy rain in one section and little or none within 2 or 3 miles or even less. There are also many other factors to be considered, account for which can be made but imperfectly. The quantity of water running off, as the author points out, will depend upon the stage of the ground-water. It, as well as the ground-water, will depend upon whether the ground is or is not frozen, the character of the soil, the amount of wooded and tillage area, and the topographic and geologic configuration of the tract under discussion.

The case in point is well illustrated by Table 2, and is even better shown by diagrams which the writer prepared in connection with his work as Resident Hydrographer at the Orford Station in 1900-04. On these diagrams the points, in different years, for maximum and minimum rainfall are not in the same relation to the points for maximum and minimum run-off in the table above referred to.

The order in amount of precipitation for January is 1902-03, 1904-05, 1903-04, 1901-02, 1900-01.

The order for amount of run-off in January is 1901-02, 1902-03, 1900-01, 1904-05, 1903-04.

The order for amount of run-off in February is 1902-03, 1900-01, 1901-02, 1903-04, 1904-05.

The order in amount of precipitation for July is 1904-05, 1900-01, 1902-03, 1901-02, 1903-04.

The order for amount of run-off in July is 1901-02, 1904-05, 1900-01, 1902-03, 1903-04.

The order for amount of run-off in August is 1901-02, 1900-01, 1904-05, 1902-03, 1903-04.

For the entire periods chosen the order in amount of precipitation is 1901-02, 1900-01, 1903-04, 1902-03, 1904-05, while the order for run-off is 1901-02, 1900-01, 1902-03, 1904-05, 1903-04.

The run-off in percentage of precipitation for the periods shown

varies more than 20% from the mean, and the largest is nearly one and one-half times the smallest. Mr. Holden.

The valuable work in stream gauging, undertaken by the United States Geological Survey, ought not to be curtailed in efficiency by reason of lack of funds for the prosecution of the work.

It is also of great importance that further investigations should be made to determine better constants for guidance in gauging rivers through the ice, and that greater effort should be made either to take the gauge heights to the water, or else to obtain more frequently the thickness of the ice under the gauging point.

Messrs. Frankenfield and Henry.

MESSRS. H. C. FRANKENFIELD* AND ALFRED J. HENRY* (by letter). —At the regular stations of the United States Weather Bureau in large cities rain gauges are generally exposed on roofs of office buildings. These exposures, while not satisfactory in all respects, are the best that can be had under the circumstances.

In winter, when roof exposures are most objectionable, the snowfall is measured in the open at some distance from buildings or other obstructions, and at a point where drifting is least pronounced.

The rain gauges at co-operative stations of the Bureau are generally located on the ground, 40 or 50 ft. distant from surrounding objects, and the exposures are considered good. The total precipitation from both classes of stations, regular and co-operative, are platted each month on large-scale maps of the United States. By this means it is possible to discover improbable values and incorrect reports. This method also affords a means of comparison between roof and ground exposures, since there is generally one or more ground-exposed gauges near Weather Bureau stations in large cities. The monthly charts of precipitation, on which the rainfall values from both classes of exposure are spread indiscriminately, show no differences of importance. The natural variation in the rainfall between any two places within a short distance of each other is much greater than any discrepancy that may appear between the catch of a roof-gauge and that of a ground-gauge.

The need of rainfall stations at high altitudes is fully realized by the Weather Bureau. A few years ago an attempt was made to increase the number of high-altitude stations through co-operation with the Forestry Service of the Department of Agriculture, but the plan failed. The Bureau now maintains, on the head-waters of some rivers, a number of rainfall stations at an average expense of $100 per annum for each station. The greater majority of its rainfall stations, however, are operated by private persons who do the work without compensation; the field of observation for this class of persons is naturally limited to the habited areas, and to those which almost without exception are found in the valleys and low levels. In the Far West and

* Asst. Chfs., Forecast Div., U. S. Weather Bureau.

Messrs. Frankenfield and Henry.

at great altitudes, the question is further complicated by the extreme difficulty of measuring precipitation when it is mostly in the form of snow and badly drifted by the winds. During the winter of 1906-07, a special form of snow-gauge was devised and placed on the summit of the Sierra Nevada and at other stations. It is expected that the experience gained by the use of this instrument will be useful in solving the instrumental problem. The great desideratum, however, is, and always will be, persons to do the work at remote places and in high altitudes. An increase in the number of high-level stations is included in a project for the reorganization of Weather Bureau work now under consideration.

In determining the mean precipitation for the several drainage basins, the author takes the average of the whole number of rainfall stations in each basin. For relatively small areas this plan works well, and the results are probably as near the truth as can be obtained with the number of rainfall stations available. The greater the number of well-placed stations, the greater will be the accuracy of the district means.

The precipitation during the cold months is more evenly distributed than that of the warm months, and, therefore, the means will be more dependable than those of the warm months. Owing to the very unequal distribution of precipitation in summer within quite limited areas, a large quantity of water may be precipitated over the headwaters of a stream, and yet none will be caught in any rain-gauge in the district. It is not advisable to draw hard and fast conclusions from either short periods of time or for small areas, but an attempt should be made to follow each and every individual case of heavy rain and to note its effect on the flow of the stream for all the varying conditions of soil and slope in the drainage area. As the author has pointed out, the run-off of one month may be almost wholly controlled by the rainfall of a previous month. It would be interesting to know how much time may elapse between the initial and the concluding effect of a heavy fall of rain on the drainage below; in other words, to note how long, whether one, two, or three months, a single heavy rain or several days of continued heavy rains will perceptibly affect the run-off of the stream below, and thus determine the general characteristics of each drainage area.

It is obvious that the manner in which the rain falls, whether as a steady rain of moderate intensity or as a heavy dash of great intensity, is of much importance in determining the relation between rainfall and run-off. In collecting the information contained in his paper, the author has performed a valuable service. It is hoped that the work may be carried on until at least 30 years' data have been accumulated, since there is a possibility that faulty conclusions may be drawn when the subject is considered in the light of a few years' observations only.

THADDEUS MERRIMAN, JUN. AM. SOC. C. E.—The author in this paper has brought together and digested in an admirable manner some of the results of the work done during the past ten years or more by the United States Geological Survey. Such an extensive compilation and comparative study of these results has, the speaker believes, been nowhere else attempted. Mr. Merriman.

The author confines his discussion to nine of the larger streams in the northeastern section of the United States, and analyzes the stream flow past fourteen stations situated at different points on these streams. From the results of these studies, he draws the general conclusion that the annual run-off from any stream in this section of the country can be predicted from the run-off data which are now available. The speaker believes that this conclusion is so broad that, even if the data on which it is based were most perfect, it could not be justified. Every stream is a law unto itself, both on account of the fact that the amount of rain which falls upon it differs from that which falls upon its immediate neighbor, and also on account of its physical characteristics, most important among which are its altitude above sea-level, its mean annual temperature, and last, but not least, its slope or steepness. All these factors combined may influence the run-off to such an extent that it is not unusual to find two streams which are within a comparatively short distance of each other and which, to all outward appearances, have the same physical characteristics; yet the yearly run-off of these streams will differ by more than 25 per cent. This is true as between the Tohickon and the Neshaminy Creeks in Pennsylvania, the Esopus, the Catskill and the East and West Canada Creeks in New York. The author's contention may be true when applied to streams having drainage areas of 5 000 or 6 000 sq. miles. On a stream of this size, the causes of difference mentioned by the speaker will average up to such an extent that run-off data may be safely used for purposes of estimating. Such large streams, however, are few; most studies and estimates are made on smaller streams, each of which has its own characteristics and must, therefore, be studied by itself. The speaker submits that the laws which govern the flow of a large stream are not applicable to the flow of a smaller stream, even if the latter be a tributary of the former.

The results obtained by the author indicate that the run-off from the streams studied is somewhat greater than has heretofore been supposed. For instance, the run-off from the Housatonic River above Gaylordsville, Conn., is shown to be 62%, while it is known that the Croton River, on the same average rainfall, will yield but 48%, this, too, in spite of the fact that the water-shed of the Housatonic is nearly three times as great as that of the Croton. Again, the percentage of run-off, as shown for the James and the Roanoke Rivers, approaches very nearly to that of the Sudbury, notwithstanding that the rain-

Mr. Merriman.

fall on the Sudbury is greater than on either of the other streams, and that it is situated in a considerably colder climate. Again, if the run-off from the Pequannock is compared with that from the Housatonic, as given by the author, it is found to be roughly 5% less, and this, too, from a water-shed which is far steeper, and on which the rainfall is materially greater.

The gaugings on all the streams discussed by the author were made by the United States Geological Survey by current-meter methods, whereas those on the Sudbury, Croton and Pequannock are the results of other methods of measurement. The precision of the results deduced from any series of observations depends on the accuracy of those observations, and it is to be regretted, therefore, that the author has not indicated his opinion of the value of the data used in his discussion.

The speaker is well aware that the methods used in the measurement of stream flow are approximate only. The unknown factors and changeable conditions which enter into them are so many that they have not as yet been co-ordinated. Practically all the stream-gauging work of the United States Geological Survey has been done by the current-meter method. The instrument used is the small Price meter, which is occasionally rated in still water. Gaugings are made from time to time at the stations where the discharge is to be measured, and, from these gaugings, a discharge curve for varying gauge heights is constructed. The gauge is then read, to the nearest half-tenth of a foot, twice daily by an observer who lives in the vicinity of the station. Most stations are located on bridges, and, at many of them, particularly across the larger streams, there are one or more piers in the river.

The accuracy of the determination of the yearly discharge of a stream measured in such a manner is dependent both on the accuracy of the measurements from which the discharge curve is determined and also on the daily observations of the gauge height; of these two main conditions that which is perhaps most likely to be in error is the former. In the readings of the daily gauge heights it is probable there will be as many plus as minus errors, and that, consequently, errors due to this cause will balance each other.

The accuracy of the method is, therefore, almost entirely, if all other conditions are the same, dependent on the accuracy of the determinations of the discharge curve, if no account be taken of the unfortunate location of gauging stations on bridges which have piers in the river and where the hydraulic conditions, consequently, are much disturbed.

The current meter of the Price type with which the discharge measurements are made is usually suspended by a cable, and the velocities for each section are determined by the point method, holding the meter at

the six-tenths depth. This meter, the speaker believes, is not well adapted to the measurement of water velocities which exceed 2 ft. per sec., in that the results given are too high, and also for the following reasons: Mr. Merriman.

(*a*).—The meter is usually rated in still water, where all particles of the water impinge on it in parallel lines. In flowing water, therefore, where the particles move in many different directions, the meter will, since it is constructed to be more sensitive in one direction than in the other, record a velocity which is higher than the mean velocity at the point where the meter is held.

(*b*).—The meter will respond to a current which impinges on it in any horizontal direction and, consequently, will record the velocity at the point where it is held, no matter whether the direction of the current is at right angles to the cross-section where the discharge is being measured or not.

(*c*).—The meter has inertia, and, therefore, will not immediately respond to a slackening of the current. This is also true as the result of its mechanical form, and, consequently, the velocity it records is greater than the mean velocity at the point where it is held on account of the pulsation of the moving water.

(*d*).—The meter, when suspended by a cable, swings back and forth like a pendulum, thus increasing the effect of the pulsations in the current.

In view of the differences between the results of the gaugings set forth in this paper, when compared with other and reliable long-term gaugings on other streams, and after having made careful allowances for the differences due to variable conditions on the water-sheds themselves, the speaker is of the opinion that there is sufficient evidence to indicate that the results for the run-off shown in this paper are uniformly too large, and that the discrepancy is approximately 10 per cent.

He believes that this excess in the yearly totals is largely due to the difficulty of measuring flood discharges with a current meter, and, since in an ordinary year considerably more than 60% of the total flow occurs at high or flood stages of the river, it becomes evident that a method to determine successfully the total yearly flow of a stream must first accurately measure the volume of water carried by a river during its high stages.

RAPHAEL ZON, ESQ.* (by letter).—The data on the relation between precipitation and run-off brought together by Mr. Hoyt are of great scientific and practical importance. Unfortunately, the figures apply to large drainage areas. As a rule, these present a complex of conditions which often compensate and counteract each other, and, therefore, obscure the real factors affecting the run-off. In many cases large drainage areas, radically different from each other, yielded prac- Mr. Zon.

* Chief, Branch of Silviculture, Forest Service, U. S. Dept. of Agriculture.

Mr. Zon. tically the same final results, from which general conclusions were often made, which could not be corroborated by observations on smaller water-sheds.

The figures of precipitation and run-off over large drainage areas, each presenting varied conditions, cannot give proper consideration to all factors affecting run-off. Such observations naturally bring out only the general factors, such as precipitation and temperature. Thus Professor A. Penk, one of the best authorities upon the regime of the Elbe, Danube, and other rivers of Middle Europe, by observing the large drainage basins of these streams, came to the conclusion that the annual discharge of rivers depends only upon temperature and precipitation; and that the ratio of the run-off to precipitation on water-sheds of radically different character remains the same, namely, the run-off constitutes about seven-tenths of the precipitation above a certain minimum (420 mm.). If the precipitation were to fall below this minimum, there would be no run-off.

Mr. Newell found about the same ratio for the drainage basins of North American rivers. According to Mr. Newell, the minimum precipitation, above which run-off takes place, is 300 mm., and the run-off is equal to eight-tenths of the precipitation above this minimum. Investigations on small catchment areas show that the relationship between precipitation and temperature, on one hand, and run-off, on the other, is not so simple. On small water-sheds where the different conditions may be accounted for, the effect of other factors is much plainer than on large drainage basins, and therefore they are apt to yield more reliable results.

Besides precipitation and temperature, the vegetative cover of the water-sheds, especially the forest cover, affects the quantity and the character of the run-off. It is a well-established fact that the quantity of water available for stream flow from forested water-sheds, all other conditions being equal, is less than from non-forested water-sheds, and it is clear why it must be so. The amount of precipitation which reaches the soil in the forest is always less than that which reaches the soil in the open, because of the interception of a part of the water by the crowns of the trees. The quantity of soil in the forest varies with the species and age of the trees, and the development of the crowns and the kind of foliage. Light rains are almost totally intercepted by the crowns without reaching the soil; the heavier the rains, the more water reaches the forest soil. The amount of precipitation which is lost in this way for the forest soil is greater in coniferous than in broad-leaved forests. It is equal to 20% of all the precipitation in a hard-wood forest; in a thin coniferous forest it is 30%, and, in a dense spruce forest, it is from 30 to 45 per cent. For this reason, coniferous forests, although they transpire less water than broad-leaved forests, demand a climate with heavier precipitation, mountain

slopes, for instance, or soils with a greater water-carrying capacity, preferably sands. Mr. Zon.

Besides the interception of precipitation by the crowns of trees, a loss of moisture on the way to the soil is also occasioned by the forest litter, which absorbs part of the water reaching the forest floor. According to Wollny's experiments, a layer of litter 5 cm. deep, composed of leaves and needles, and exposed to sun and light, allows only from 64 to 70% of all the precipitation that falls upon the forest floor to reach the soil, and only 53% reaches the soil through a layer of moss of the same depth. Thus the forest soil receives the least precipitation, next comes meadow land, and lastly tilled land.

In the forest, only the upper layer of the soil is moister than in the open, the lower layers being always drier. Thus, according to Ebermayer's experiments of 1884-86, the percentage of soil humidity at different depths in an uneven, aged, spruce forest and in the adjoining field is as shown in Table 7.

TABLE 7.

Depth.	Young forest, 25 years old. Percentage.	Middle-aged forest, 60 years old. Percentage.	Mature forest, 120 years old. Percentage.	Field. Percentage.
0- 5 cm.....	28.97	20.48	40.25	22.33
15-20 "	19.19	19.07	19.29	20.62
30-35 "	19.10	16.07	18.29	20.54
45-50 "	18.39	16.26	20.22	20.24
75-80 "	18.00	17.90	21.12	20.53
Average....	16.67	17.32	19.73	20.23

The middle-aged forest always produces the greatest desiccation of the lower layers of the soil. This is especially noticeable in summer and fall. In such a forest the soil humidity at a depth of 40 cm. is, on an average for the summer, 15.12%, while in the field at the same depth it is 19.89 per cent. The dryness of the soil under the forest is caused by the transpiration of water by the forest. Since trees produce more organic substance per unit of area than other plants, their expenditure of water for the formation of cells, therefore, must also be greater. The enormous transpiration of water by the forest and the smaller amount of precipitation received by the forest soil cause the greater dryness of the soil under the forest.

All this goes to show that the forest cover tends to decrease the total quantity of water available for stream flow. A comparison of forested and non-forested water-sheds, especially in the South, would probably show a smaller total amount of run-off from forested, than from non-forested, water-sheds. It would also show, however, a dif-

Mr. Zon. ference in the distribution of the run-off throughout the year, and throughout a cycle of dry and wet seasons. In all cases, the forest tends to decrease the difference between the maximum and minimum run-off, and to maintain a steady flow of streams. This is very clearly shown by Professor Toumey's investigation in the San Bernardino Mountains of Southern California. Here the run-off during the dry summer months was maintained in the forested areas, while, over an unforested area, under the same conditions, there was practically no run-off during the summer. The data concerning precipitation and run-off, as brought out by Mr. Hoyt, while true for average conditions, such as exist over large drainage basins, may not be applicable to smaller streams and water-sheds where there is a lesser variety of conditions which compensate each other, and, therefore, other factors, besides precipitation, are at play.

Mr. Vermeule. C. C. VERMEULE, ESQ. (by letter).—While there has been accumulated a large addition to the records of stream measurements in the United States during the past ten years, the writer has not yet found it possible to ignore the effect of differences of rainfall, temperature, cultivation and soil conditions in computing the run-off of any stream for which gaugings do not exist and for which he finds it necessary to have the most accurate estimate of run-off which can be made.

The differences in run-off of streams in the same region are so great that to apply directly the gaugings of one stream to some other stream, without correction, must be regarded as unscientific and unsatisfactory. If we have not at hand accurate measurements of precipitation upon the two streams covering a considerable number of years, we may be unable to do better, but with measurements of precipitation and of temperature, and an examination of the respective water-sheds, carefully noting differences in soil conditions, topography, forestation and cultivation, the hydraulic engineer of experience should be able to make a much closer approximation to the run-off by properly correcting a set of gaugings to allow for these differences. Such correction can only be made after one has familiarized himself with the phenomena and the laws which govern the relation between rainfall, temperature, topography, soil conditions, and the run-off of streams. A patient and careful analysis of such data has brought the writer to the firm conviction that the greatest need is that certain typical streams throughout the United States, comparatively limited in number, should be gauged with a high degree of accuracy, with simultaneous rainfall measurements at a number of well-selected points upon their water-sheds, and that these gaugings should be continued over a considerable period. The same money expended in gauging a very large number of streams in an indifferent manner, without careful simultaneous rainfall records, if expended upon more

careful measurements of one-tenth of the number of streams selected as typical water-sheds would do far more to reduce hydrology to a science. Mr. Vermeule.

In studying the relation between rainfall and run-off, it is necessary to keep in mind the fact that the run-off is merely the difference between the rainfall and the evaporation taken in its broad sense to include the water taken up by vegetation as well as that directly evaporated in the atmosphere. Under ordinary natural conditions, the water which percolates downward and does not reappear through springs is insignificant in amount. Evaporation obeys well-known laws which have been the subject of much study and investigation, so that there is much at hand on which to base a reasonable method of computing it. It is to a large extent a function of the temperature. Having the rainfall for a given period, if the evaporation for the same period could be computed, the difference should represent the run-off, and it will be found to do so, nearly, if the period in question begins and ends with the free ground-water at substantially the same level. In actual experience, however, it will be found that, with the same amount of rainfall, the run-off may vary in different years from about 37% to about 63% of the rainfall, or as much as 10 in. per annum. Under such conditions, the scientific mind cannot be satisfied to proceed with any rule-of-thumb method, or with any method which does not allow for such variations. This particular difference is due to the water taken up by the ground and stored therein, to be fed out to the streams later during the dry season, and it illustrates the very important effect of soil and sub-soil conditions in maintaining the dry-season flow and equalizing the yield of the stream.

About 1890 it became the writer's duty to make as accurate an estimate as possible of the run-off of the streams of the State of New Jersey. This, at that time, had become urgently necessary, and there was no time for extended gaugings of streams. Gaugings were made to a very considerable extent during the four years of work, but much reliance had to be placed upon the longer series of measurements then available upon the Sudbury, Connecticut, Croton, Tohickon, Neshaminy and Perkiomen. Through the courtesy of the Dundee Water Power and Land Company, the writer was also fortunate enough to obtain a 17-year record upon the Passaic. A study of these records showed a marked resemblance between the Sudbury, Croton and Passaic, so striking on close analysis that it was found that these three streams exhibited a quite well-defined relationship between rainfall and run-off. After plotting the results, a formula was deduced to cover this relation, but it soon became evident that the data exhibited by these measurements could not be used as applicable to other streams in New Jersey without correction. This led to the application of a correction, due to differences of evaporation, which was found to in-

Mr. Vermeule.

crease and decrease with the temperature, and a further correction which followed for differences in topography and soil conditions, or, in other words, the varying effect of ground storage.

Later, in 1899, while investigating the effect of forests upon stream flow, the writer utilized additional data then available corvering thirty-four streams in the Eastern United States, extending from the Merrimac on the north to the Savannah on the south, together with a few in the Mississippi Valley and four in England. Giving these results weight in accordance with the length of the record and the known reliability of the rainfall and run-off measurements, a more accurate formula for computing the evaporation from the rainfall was deduced.* The results given by this formula agreed with the observed results upon seventeen streams having an average length of record of 12 years, in all cases within 5%, the mean error being only 1½ per cent. Three more streams, with records averaging 11 years, vary from 6 to 8%, and the remaining fourteen streams, having shorter records, averaging 6.5 years, show wider differences, the average discrepancy being 5 per cent. These results, together with later experiences, have convinced the writer that it is possible to evolve a formula and method of computing run-off from rainfall which, generally speaking, will give results far more reliable than can be obtained either from adopting figures of run-off on other streams without correction, or from gaugings upon the stream in question if they do not extend over a longer period than 5 or 6 years. One has only to examine any reliable record, such as that of the Sudbury, in order to be warned that gaugings which extend over only one or two years may vary so widely from average experience upon any given stream as to be not an aid, but possibly a hindrance, to good judgment. Short series of gaugings may be very valuable, if used properly, that is, by taking into account the rainfall during the period, together with the average rainfall, and the rainfall for the driest years upon the same water-shed. Here, again, one must know with reasonable accuracy the true relation between rainfall and run-off.

Personally, the writer has never found much use for percentages in the computation of run-off. If percentages are determined from average rainfall and run-off, they are only applicable to a fictitious average year, or a year which never actually occurs in Nature. Examine any extended run-off measurements, and it will be readily discovered that, during many months in which little or no rainfall occurs, the stream will continue to discharge a considerable quantity of water. This makes it evident that there is no direct relation whatever between the run-off in any given month and the rainfall for that month. This does not refer to winter when the precipitation is locked up in snow, but to the dry hot months in summer. For water-power pur-

* Geological Survey of New Jersey, Annual Report for 1899, p. 148.

poses the writer has of late been able to obtain excellent results by taking gaugings covering a long period upon a stream on which the topography and soil conditions are similar to the one for which he is estimating, and determining by actual count the number of days during which this stream stood at or above given stages of flow, and then plotting the results in a diagram. Having done this, he determines as accurately as possible, from the records of the United States Weather Bureau, the average rainfall and temperature upon this stream during the period covered by the gaugings, and then determines the same data for the stream to be estimated; next, he computes the evaporation and run-off for each stream, and increases or decreases the stages of flow in the proportion that the run-off upon the measured stream bears to the run-off of the stream to be estimated, plotting a corrected curve for the stream to be estimated. Where this method has been adopted and afterward compared with the records of water-power owners upon the stream, a close agreement between the two has been found. Mr. Vermeule.

The accurate computation of run-off is useful mainly to determine the amount of storage needed to maintain a given yield, or the amount of water-power to be obtained upon a given stream. To some the whole subject may seem to be largely academic, but, not only is it of importance for the purposes indicated, but the relation between rainfall and run-off is of general scientific interest, and bears upon the subjects of forestry, sub-soil drainage, irrigation, etc. While it is a study which requires more time and closer application than can usually be given to it by an engineer in active practice, this is the best reason why we should the more readily encourage every attempt to elucidate the subject and to simplify and improve the resulting formulas and conclusions.

The time when there will be at hand reliable and adequate gaugings of each stream one is called upon to develop is still far distant. Meanwhile, there is need to know how to compute scientifically from the meteorological data now so generally available.

H. K. Barrows, Assoc. M. Am. Soc. C. E.—The speaker wishes to comment briefly on several points in Mr. Hoyt's interesting and instructive paper, especially on the matter of accuracy of methods in use by the United States Geological Survey. Mr. Barrows.

The Geological Survey has given much attention to the accuracy of both field and office work in stream gauging, and every effort, consistent with the general object of this work, has been made to obtain data relating to or confirming its accuracy. Some of this information has appeared in the Water Supply Papers, already published, while many additional unpublished data have been obtained by the various district hydrographers.

Previous to 1906 the progress reports of stream measurements contained no comments regarding the accuracy of estimates of flow. In

Mr. Barrows. the report for 1906, however, an attempt has been made to ascribe to each gauging station an accuracy factor—in many cases varying with the time of the year. The general remarks relating to this factor are:*

"In order to give engineers an idea of the relative value of the various data, notes in regard to accuracy are given as far as possible. This accuracy depends on the general local conditions at the gaging stations and the amount of data collected. Every effort possible is made to so locate the stations that the data collected will give a high degree of accuracy. This is not always possible, but it is considered better to publish rough values with explanatory notes rather than no data.

"In the accuracy notes the following terms have been used, indicating the probable accuracy in per cent. of the mean monthly flow. As these values are mean values, the error in the value for the flow of any individual day may be much larger.

"Excellent indicates that the mean monthly flow is probably accurate to within 5 per cent.; good, to within 10 per cent.; fair, to within 15 per cent.; approximate, to within 25 per cent."

Mr. Merriman especially criticises the current-meter gaugings of flow and the results obtained with the small Price current meter, but agrees that the unbalanced errors of gauge-height observation in monthly mean discharge will be small. In the speaker's opinion, the current-meter gaugings—and therefore the discharge-rating curves—constitute the most reliable part of these data. Mr. Hoyt has shown one example of these rating curves, from the great number that exist, and it is quite unnecessary to call further attention to the remarkable consistency of the current-meter measurements in their relation to the discharge-rating curve.

What the absolute errors are can only be determined by comparison with some other standard of measurement—sharp-crested weirs, measuring tanks, etc.—but, thus far, such comparisons as have been made indicate that a carefully made current-meter measurement at the ordinary gauging station will give results far within the degree of accuracy required in such work.

At one of the gauging stations of the Geological Survey, *viz.*, Little River, at Blandford, Mass., it has been possible during a portion of the past year to maintain a sharp-crested weir just below the regular station, thus enabling a direct comparison of results by these two methods of measurement. These are given in detail in Water Supply Paper No. 201, pp. 105-110, but a summary of results by months will be of interest, and is given in Table 8.

While the daily discharge by these two methods exhibits a considerable percentage of variation at times, the monthly means, and even the weekly means, are remarkably close in agreement. Furthermore, this gauging station is what would be classed as only a fair station—the stream bed being very rough. The differences in this

* Water Supply and Irrigation Paper No. 201, p. 12.

case are evidently mostly due to errors or inaccuracies in gauge readings. It will be noticed, further, that the differences at some times are positive, at other times negative. Mr. Barr

TABLE 8.—MONTHLY MEAN DISCHARGE OF WESTFIELD LITTLE RIVER, NEAR BLANDFORD, MASS., 1906, AS DETERMINED BY WEIR AND CURRENT-METER RATINGS.

Month.	DISCHARGE IN CUBIC FEET PER SECOND BY		Percentage of difference as regards discharge by weir.
	Weir.	Current meter.	
August (10–31)	12.3	13.0	+5.7
September	9.1	9.0	−0.9
October	42.2	42.9	+1.7
November	55.7	57.6	+1.6
Average for period	.....	.....	+2.0

Mr. Hoyt's paper deals solely with large drainage areas where the average rainfall is fairly well determined. Information is frequently needed about rainfall and run-off from small drainage areas—perhaps from 10 to 50 sq. miles—such as frequently have to be considered in procuring a municipal water supply. In such cases the storage area of tributary ground-water may depart considerably from the topographic limits of the drainage basin and markedly affect the low-water flow per square mile. Local areas of high average rainfall may be included—this being especially common upon mountain slopes—so that the run-off per square mile will be much higher than with small drainage areas under ordinary conditions. This last condition is well illustrated by the run-off of small drainage basins in the White Mountain region, New Hampshire,* noticeably that of Israel, Ammonoosuc and Zealand Rivers.

The storage of water by lakes or reservoirs in the drainage basin, if of considerable amount, will modify the run-off in monthly distribution, and may also in the total amount for the year on account of evaporation of the water while being held.

Table 9 shows the rainfall and run-off data relating to the Kennebec River and Cobbosseecontee Stream, Maine.†

The Kennebec River, at Waterville, has a lake surface of about 5% of the total drainage area at that point, and, of this amount, only a little more than one-half is utilized for storage purposes.

* Water Supply Papers Nos. 97, 124, 165 and 201; also reports of the New Hampshire State Forestry Commission, 1903-06.

† Water Supply Paper No. 198.

Mr. Barrows. Cobbosseecontee Stream has about 9% lake surface, nearly all of which is utilized for storage. While the total run-off for the year is less for Cobbosseecontee Stream, as indicated in Table 9, yet the monthly means do not fluctuate as much, and the unit flow during the low-water season is always greater.

TABLE 9.—RUN-OFF, IN INCHES, ON DRAINAGE AREA, FOR THE PERIOD GIVEN; ALSO RAINFALL, ETC., ETC.

Period, etc.	Kennebec River, at Waterville, Me., 1893–1905, inclusive; 4 270 sq. miles.	Cobbosseecontee Stream, at Gardiner, Me., 1891–1905, inclusive, 240 sq. miles.
January	0.77	1.13
February	0.77	1.11
March	2.34	3.19
April	5.90	3.03
May	4.96	1.81
June	2.44	1.20
July	1.88	1.08
August	1.04	1.13
September	0.78	1.00
October	0.80	0.91
November	1.09	0.83
December	0.93	0.97
Run-off, for year	23.70	17.39
Rainfall, for year, in inches	39.63	42.18
Percentage of run-off, for year	0.60	0.41
Total loss, for year, in inches	15.93	24.79

Mr. Stevens. J. C. STEVENS,* ESQ. (by letter).—It is very gratifying indeed to see the subject of the relation of rainfall to run-off treated so comprehensively and in such a systematic manner as in Mr. Hoyt's very timely paper. It is to be hoped that other sections of the country will speedily be taken up in a similar manner, and that all available data will be presented. Until this is done, it is doubtful whether the engineering profession will come to a full realization of the almost utter futility of predicting the behavior of streams from precipitation records. Thus far, the only law which has been discovered is that a high rainfall produces a high run-off, and a low rainfall a correspondingly low run-off. "High" and "low," and "more" and "less" are about as concrete terms as one dare apply to this relation. This is not at all surprising when one considers the many factors that must be taken into account, and of which absolutely nothing is known. There is not even a notation which can express the relative values of such factors as soil conditions, topography, forestation, etc., in respect to which every drainage basin is different, but is greatly affected thereby. Taking long periods of time, a large number of well-distributed precipitation stations and reliable run-off data, it is sometimes possible to show a rather poorly defined relation, but to attempt any determination of the

* District Hydrographer, U. S. Geol. Survey, for Washington and Oregon.

rate of flow, or a quantitative determination for short intervals of time, is almost absurd. Moreover, in every relation thus far determined there are always certain inconsistencies which perhaps are not without explanation when once recognized, but their causes are so obscure as never to have warranted their prediction. In almost every Western State may be found monuments to some engineer's (not infrequently county surveyor's) misguided views on this question, in the shape of irrigation projects involved in endless litigation and expense, wholly abandoned, or necessarily abridged on account of the non-fulfillment of the predicted behavior of a stream. Mr. Stevens.

Until F. H. Newell, M. Am. Soc. C. E., Director of the United States Reclamation Service, in the early Eighties, devised the method now in use of calibrating or rating a favorable cross-section of a stream by current-meter measurements, there was no inexpensive method of obtaining run-off. Since that time the method has been improved and systematized, until to-day the records of stream flow obtained by the United States Geological Survey are generally accepted as being authentic and reliable as far as it is possible to obtain them with a rational expenditure of time and money. In the West an engineer's office is incomplete without a current meter, and the methods of making measurements and preparing the data developed in the Government service are largely followed by those engineers in their private practice.

The subject of hydrography is perhaps as completely a scientific study in approximations as any with which the engineering profession has to deal. The absolute accuracy attainable in any particular case depends wholly on the amount of funds available, while the degree of accuracy required depends on the use the data are to serve. Rarely will a subsequent variation of 10 or 15% from the basic data used prior to construction involve a plant in litigation or necessitate material alterations, while often a more liberal allowance is permissible. The demand upon the Geological Survey for run-off data all over the United States has made it necessary to include as many streams as possible in the investigations. In view of this fact it has been impracticable to adopt ultra-refined and expensive methods for general use, as their adoption would not be consistent with a rational expenditure of time and money.

For example, in the Columbia River District (Oregon and Washington), in 1906, the United States Geological Survey maintained 86 regular gauging stations where rating curves were developed by current meter. At these stations 719 discharge measurements were made, resulting in the publication of 753 monthly values of run-off. This was done at a total cost of $10 974, a cost of $127.60 per station, or only $14.60 per "monthly mean." Where conditions were favorable, the published results are probably within 10 or 12% of the truth, at others the results are approximate, while, at some, estimates were im-

Mr. Stevens.

possible. If accuracy had been the only essential, and results within 4 or 5% had been demanded, the entire appropriation would have been required for two or three stations, which would have defeated the entire purpose of the organization, which is that of benefiting the greatest number.

It seldom happens that the mean run-off of a stream will satisfy the engineer's demand for data, yet this is the only quantity determinable with any show of reason from precipitation records. Irrigation, municipal supply, and particularly water-power development require a knowledge of the time distribution of the various rates of flow. But it must be conceded that this feature from rainfall alone is wholly indeterminate. A single example will suffice. In Nebraska the Republican River has a drainage area of 22 000 sq. miles, and receives a mean annual rainfall of 22.12 in. The Loup River, with a drainage area of 13 540 sq. miles, receives a mean annual rainfall of 22.70 in. Winter records of run-off are not available, but 10-year means for the period from April to October, inclusive, show that the Loup discharges nearly 10% of the rainfall during that period while the Republican discharges less than 2% of the precipitation during the same time. These two river systems exist nearly side by side under practically the same climatic conditions. Neither area is forested, and the topography of each is similar. The difference in discharge is due to the difference in soil and other geologic features; yet it would have been impossible for anyone to have predicted this difference. It would have been the easiest matter in the world to have made an error of 500% in the mean open-season run-off of the Republican River, and yet this mean run-off is the only quantity susceptible of rational determination from rainfall records. Any attempt to obtain a minimum rate of flow in any other manner than by actual gaugings would be wholly absurd, for, notwithstanding the fact that the topography, climatic conditions and rainfall are practically the same for the two areas, the minimum rate of flow of the Loup River is about 1 000 cu. ft. per sec., while the Republican goes entirely dry.

Since, then, it must be admitted that the essential part of run-off data is wholly indeterminate, and that only mean annual values are at all possible from records of rainfall, is it not better to depend wholly on gaugings, especially when results sufficiently accurate for all practical purposes are obtained at such reasonable expense?

Mr. Leighton.

M. O. LEIGHTON,* ASSOC. M. AM. SOC. C. E. (by letter).—The criticisms and suggestions that have appeared in response to Mr. Hoyt's paper are heartily welcomed by those who are responsible for the conduct of the investigations of the Geological Survey on the water supply resources of the United States. It is somewhat dis-

* Chief Hydrographer, U. S. Geological Survey.

appointing that some of the members who, in the past, have made very helpful verbal criticisms have not transmitted the same to the Society for publication in connection with this paper. Some of the criticisms that have been recorded will be utilized by the Geological Survey at once. It is well known that those who give exclusive attention to the details of any subject frequently fail to take a position far enough away to enable them to get a good perspective of the whole field, whereas those on the outside fail to take in much more than the broad perspective. This condition is made apparent in some of the discussion upon this paper, but more especially by the criticisms that have been received in times past from eminently qualified engineers. Therefore, it is only by taking into consideration both the nearer and the broader views that it can be hoped to bring the work up to its maximum efficiency. Mr. Leighton.

There is no derogation in the statement that the majority of eminently qualified engineers have wondrously erroneous ideas concerning the controlling conditions in river-flow measurement. This is not surprising, because the practical measurement of stream flow involves specialization, which the engineer in broader practice cannot profitably keep in mind. This explains why the Geological Survey has received advice from unquestionably high sources which has revealed an almost complete lack of information concerning even the mechanical details of practical river-flow measurement. It is not uncommon to see reports, emanating from such sources, in which the run-off data have been refined to the third and fourth decimal place, while the methods of gauging the rivers in question have unquestionably been inaccurate by 5 and sometimes even 10 per cent. The Survey has seen engineers insist upon gauge readings down to hundredths of an inch when the chains with which the measurements were made would vary several hundredths within the ordinary range of temperature. Also, engineers have insisted upon minute accuracy in stream gauging, when, in the practical installation of power appliances, it is not possible to approach within a wide margin of such accuracy. Such incidents are to be expected, and therefore no complaint is made concerning them, and the statements are inserted here merely to call attention to the tendency.

Several of those who have taken part in the discussion have called attention to the fact that the run-off data presented in Mr. Hoyt's tables apply to so wide an area that complex conditions are presented which compensate and counteract each other and thereby obscure the real factors affecting the run-off. No one appreciates this better than Mr. Hoyt and the writer, but they are obliged to face the condition that in drawing conclusions from small drainage areas they merely accentuate the errors arising from unequal distribution of rainfall and of the effect produced on run-off by variable conditions of the

Mr. Leighton.

land surfaces. Messrs. Frankenfield and Henry state very truly: "It is not advisable to draw hard and fast conclusions from either short periods of time or for small areas." Farther on, they say:

"It is obvious that the manner in which the rain falls, whether as a steady rain of moderate intensity, or as a heavy dash of great intensity, is of much importance in determining the relation between rainfall and run-off."

Now, it appears to the writer that the fact that Mr. Hoyt's data cover wide areas of drainage is, in a way, a distinct advantage, because local errors on either side of the line of accuracy are thereby compensated. Of course, this would not be true if there were accurate measurement of rainfall, representative of these small areas, but this would be impossible under the present appropriation of the Weather Bureau, and therefore it has been the experience of the Geological Survey that it is far nearer the truth to consider the matters over a broad area, in which local inaccuracies of reading and peculiarities of condition are merged.

There is no contention that the methods and plans of work used by the Survey are the best that could be instituted. It is freely admitted that the country would profit by a minute and detailed study of the subject on all drainage areas, large and small; but, this is impossible of accomplishment with the funds at the disposal of the Survey. The practical question, then, is as follows: Given a sum of money not sufficient to carry on the work in an ideal manner over all the country, what is the best use that can be made of it? Shall it be expended on a minute study of a few streams whereby results may be secured which shall be ideal for those particular streams, but not necessarily applicable to others; or shall the endeavor be made to secure with reasonable accuracy—as reasonable at least as the present practical demands of water development require—a system of useful data for the entire country? Take any river that has been studied by the Geological Survey, and the amount of money that has been expended in such study: To make an investigation that would conform to the ideas of some of the critics would require from twenty to thirty times the amount of money actually expended. Assume, now, that this detailed work has been accomplished, and place it side by side with that which the Survey has actually performed. The former is better, of course. It satisfies the demands of precision and gives to the engineer who has performed the work, or to the one who uses it, the comfortable and much-to-be-desired feeling of sufficiency. But, applying both sets of results to an actual power installation, of how much more value is the one than the other? Does the one enable the engineer to adjust the capacity of his wheel unit or the size of his proposed storage reservoir by a degree of efficiency that is commensurable with the difference in the cost of the work? The Survey, after studying the matter for about fifteen years, believes not.

Mr. Leighton.

There is another and very practical consideration in connection with the policy in this matter which oft-times escapes the engineer in practice. It should be remembered that this work is done under National appropriation, and presumably all parts of the country must receive an equal degree of attention. If all the money were expended on a few streams, it is clear that this exclusion of the remainder of the country must continue for a long period of years, and that no data would be available except on the small proportion that is actually under study. Therefore, is it not better to present to the entire country a series of results which can be used with satisfaction, even though they be not as good as could be desired, than it is to deprive the greater part of the country of useful results? The Survey believes it is better.

The writer is especially glad to read the discussion by Mr. Vermeule, whom he regards as one of the pioneers in this line, and whose work he has long considered with admiration. Mr. Vermeule contends that it would be better to confine the work of the Survey to a number of streams and insist upon minute accuracy. There is no doubt that, if this were done, he is correct in his statement that it would do far more to "reduce hydrology to a science." In making this statement, Mr. Vermeule evidently loses sight of the fact that under the present appropriations of the Federal Government the object of the work cannot be to reduce hydrology to a science, but, on the contrary, to give to the people of the United States reasonably accurate information concerning stream-flow so that they will have something upon which to work, where before they had nothing. However praiseworthy his plan may be, it is futile to consider it with an appropriation which in the past two years has been reduced one-half.

An important matter which should be brought out in connection with Mr. Vermeule's description of his study of the relation between rainfall and run-off is his use of evaporation records. He states:

"Evaporation obeys well-known laws which have been the subject of much study and investigation, so that there is much at hand on which to base a reasonable method of computing it."

Recent, but yet unpublished, work of the United States Weather Bureau has shown that our ideas of the laws governing evaporation and the methods of estimating it are so much at variance with the truth as to make it extremely doubtful whether any of our previous work on this subject is worthy of consideration. The Weather Bureau, assisted by the United States Reclamation Service and the United States Geological Survey, is planning to devote a large amount of time and money to the most extensive investigation of evaporation that has ever been made, the seat of the experiments being the Salton Basin, in Southern California, which, as everyone knows, was recently submerged by reason of a crevasse in the Lower Colorado River. In planning for this work, it was found necessary to investigate

Mr. Leighton. methods, and the entire past season has been devoted to this, with results that throw doubt on nearly every feature of our past studies of evaporation. In fact, it is necessary to start work in the Salton Basin as though studies of this character constituted a new and untried field. Taking this into consideration, in connection with the indisputable facts concerning present inaccuracies in rainfall estimates, it is difficult to appreciate how a calculation of run-off with these two factors can be as useful as the present methods of the Geological Survey.

One word more, with regard to criticisms that have been made concerning specific stations maintained by the Survey: The fact that the Survey maintains and publishes records of any station does not commit it to any expression of belief in the accuracy of those records. It is not always possible to determine beforehand how good the records from any station may be. Sometimes it requires a full year to show whether a station is or is not worth the maintenance. In case such a station is found to be poor, the results are published merely for what they are worth, and it is the aim to give such an interpretation of them in the reports of the Survey. Other stations are established in places where it is known that the records will not be first-class, but where there is a strong demand for the best that can be produced. Strange as it may seem to those who have never gone into the details of river-station maintenance, there are many streams in the United States along the entire courses of which there is not a permanent section suitable for accurate work. The only thing that can be done under such circumstances is to produce the best possible with the funds available. With this explanation it is confidently hoped that some of those who have had occasion to criticize the Survey for the maintenance of certain stations will take all the facts into consideration, and will not grade the entire efficiency of the service on the basis of such conditions.

Mr. Newell. F. H. NEWELL,* M. AM. SOC. C. E. (by letter).—With reference to the discussion on rainfall and stream flow, there is one statement made by Mr. Thaddeus Merriman which should not go without some challenge. He states:

"In view of the differences between the results of the gaugings set forth in this paper, when compared with other and reliable long-term gaugings on other streams, and after having made careful allowances for the differences due to variable conditions on the water-sheds themselves, the speaker is of the opinion that there is sufficient evidence to indicate that the results for the run-off shown in this paper are uniformly too large, and that the discrepancy is approximately 10 per cent."

After going over the line of reasoning indicated in this statement,

* Director. U. S. Reclamation Service.

the writer is unable to agree with the generalization, as it apparently ignores the results of experience of many men who have devoted the best part of their lives to the building up of this work. Great care has been taken in securing the highest obtainable accuracy, compatible with the necessity of having, at small cost, data from widely scattered rivers. Several of the best mechanicians of the country have contributed in the effort to perfect the devices for measuring the velocity of the water. These devices have been checked again and again against other methods of obtaining the rate of flow. It is certainly a surprise to this large group of well-trained men to be told that, in spite of the many precautions taken, the results are uniformly 10% too large. Mr. Newell.

Without entering into the details of the tests which have been made, in various parts of the country, of different methods and devices used in checking the results, it is sufficient at this time to state that, among the large body of experienced men who have devoted years to work of this character, there will be found no general admission that it is in error to this degree.

A. P. Davis, M. Am. Soc. C. E. (by letter).—This paper presents in small compass a remarkably large quantity of valuable information, and although the drainage areas represented are large, the results can be applied with considerable value to smaller areas within the basins themselves and in adjoining basins, provided due consideration is given to all physical differences which affect run-off. Mr. Davis.

The discussion has developed a wide difference of opinion regarding the relative value of stream gaugings and run-off results computed from rainfall, temperature, and other physical data, and, with some of the opinions expressed, the writer cannot agree.

While his experience with streams in the East is less than with those in the West, it is sufficiently extended to convince him that the only excuse for depending entirely upon rainfall and other physical data for computing run-off is the fact that results must be obtained in too short a time to admit of stream gauging. If works must be planned on short notice, and, say, only a year can be given to the study of the water supply, the best possible use that can be made of that year is to measure the stream at the points where its discharge is desired.

These measurements, when interpreted in the light of existing rainfall and run-off data, showing their relation to adjoining areas and to other periods, and when corrected for such physical facts as can be ascertained in the same time, will give more reliable results than any based on purely theoretical considerations, which omit or ignore the actual measurements of the stream.

Mr. Vermeule appears to believe that he can compute run-off from

Mr. Davis.

long rainfall records with an average error of 1.5 per cent. It is safe to say that the probable error of observation of rainfall records themselves is much greater than this. When, to this, is added the fact that rain falls in spots and streaks, and is to the last degree erratic, from many causes, it is safe to say that the annual rainfall on any considerable basin has never been and cannot be measured (with practicable means) within a much larger percentage than that mentioned. Add to this the unavoidable errors which creep in, due to the multitudinous physical details which affect run-off, and the influence of which cannot be accurately known, and what becomes of the belief in such close results from computations?

Practically nothing is known of the laws of run-off, except as deduced from an intelligent use of actual comparisons of measurements of rainfall and run-off; and the modifying circumstances of slope, topography, temperature, wind movement, manner of rainfall, and many others, are so numerous, so difficult to ascertain, and modify each other in so intricate a manner, that one can never hope to reduce this theory to an exact science.

The chief value of hydrographic data is their bearing on the future, enabling one to approximate probable future results. As no stream ever discharges the same amount in consecutive years, and no rainfall record for any year ever agrees with that of any other year, the need of precise measurements is not apparent where the cheaper, more extensive, and less precise measurements are far within the necessary error of results possible to deduce from them.

The Geological Survey has wisely adopted the policy of making a large number of measurements with an ample degree of accuracy, rather than a few measurements carried to expensive refinements of no value in their influence on final results.

The Reclamation Service has had occasion to develop a very high appreciation of the value of the large number of stream measurements made by the Geological Survey on western streams.

Mr. Grover.

N. C. GROVER, ASSOC. M. AM. SOC. C. E. (by letter).—Mr. Hoyt has shown the futility of attempting to estimate closely the run-off from the rainfall. The writer does not understand why engineers continue to use this method of estimating run-off where actual measurements of discharge have been made and are available. There is, however, as shown in the discussion, a legitimate use for long-time records of precipitation to supplement the short-time records of run-off, and in estimating the approximate run-off from basins in which no measurements of discharge are available.

The accuracy of the records collected by the U. S. Geological Survey has been brought in question, and a plea made for a few gauging stations maintained in such a way as to obtain an extremely accurate record at each, instead of the much greater number of stations main-

tained at present. The accuracy of the records has been attacked, without presenting the data on which the attack has been based and apparently without taking into consideration the greater proportion of water areas in some basins than in others, and, in fact, without considering any of the factors which are known to affect run-off. The dangerous assumption is also made that records collected at other than current-meter gauging stations are correct, and that all discrepancies between records for different stations are due to errors in current-meter work. A reasonable degree of inaccuracy in meter records will be admitted, of course, by all, and recent publications of the Survey contain statements relative to the probable accuracy of the work at each station. In discussing the value of increased accuracy for such records, however, one should consider the greater cost of the more accurate records, and also the extreme variability of run-off from day to day, month to month, and year to year, as well as the use which is to be made of the results. An examination of any long-time record of run-off will reveal at once the fact that the extremes of annual discharge vary from 25 to 50% from the mean, and that in not more than one year in five will the yield be within 5% even of the mean. The monthly records show much wider variations, which occur without order or sequence, and cannot be predicted. When it is considered that there is no possibility of forecasting, even approximately, variations in rainfall and run-off which will occur in the next year or the following, and, further, that the principal use of records of run-off is to serve as a basis for predicting future yields, the force of the plea for extreme accuracy is lost. Mr. Grover.

While the writer appreciates the scientific value which the proposed accurate measurements would have, if they were accompanied by equally good records of precipitation, temperature, and other climatic conditions, and an exact record of the geology, topography, vegetation, etc., of the basin, and, especially, if such accurate records pertained to basins in which the conditions affecting run-off were uniform throughout the area, he does not believe that such records for a few stations could have the practical value of the much larger number of less accurate records which may be collected for the same total cost. For practical uses, the collection of extremely accurate records of run-off would be comparable with the application of the method of surveying valuable city lands to the much less valuable forested areas.

The writer, therefore, believes that the practical value of the paper and its data should not be obscured by a discussion of fine points of accuracy which are not of especial value when applied to records of this class.

F. W. Hanna, Esq.* (by letter).—The growing utilization of the water resources of the country is rendering imperative the need for accurate and extensive data on stream flow. The reversion from Mr. Hanna.

* Engineer, U. S. Reclamation Service.

Mr. Hanna. steam to hydraulic power, the use of streams for water supplies and the disposal of sewage, the development of irrigation, and the protection of municipal and agricultural property along many of the important rivers of the United States are daily making stronger demands for just such information as has been brought together in Mr. Hoyt's paper. The river discharge data and the rainfall data used in compiling Mr. Hoyt's tables have been obtained by and under the direction of experts in the employ of the Government. The field and office methods practiced by these experts have been developed through years of careful thought, and, therefore, are about as reliable and accurate as the uses of hydraulic data require. Such data are necessarily particular, not general, in character, and, in extending their application, the engineer must look closely into similarity of conditions.

If an engineer draws conclusions as to the definite run-off of a river by considering the available rainfall, evaporation, and plant interception and transpiration data on its drainage area, in conjunction with similar data for the drainage area of a stream with a known run-off, he takes great chances on the soundness of his conclusions. In the determination of rainfall data, the measurements at the gauging station are not exact, and very much less so must they be when applied to a large drainage area with a varying topography. Evaporation data are still less satisfactory. Very few extensive experiments have been made along this line, especially in connection with evaporation from soil. The greater portion of any river drainage area is composed of land surfaces comprising nearly every class of soil, and covered by many kinds of vegetable growth. The quantity of water disposed of by plant interception and transpiration is yet largely in the field of the unknown. Accurate estimates of stream flow from such conditions, therefore, are impossible. Inasmuch as such a method of procedure in determining stream flow would involve the substitution of results of unknown accuracy for those of known accuracy, it does not seem to be desirable to adopt it and to curtail the gauging of streams to a few widely scattered rivers.

The Price current meter, as well as all other hydraulic instruments used for measuring the flow of rivers, has inaccuracies. For all practical purposes, the inaccuracies of this particular instrument are negligible. Owing to the sensitiveness of motion of the wheel in the direction of rotation, the Price meter may not give the actual velocity in a cross-section at right angles to the channel if the water filaments at the position of the meter are not approximately parallel to each other and to the channel. But, although it is well understood that all the water filaments in the whole measuring section of a river are not parallel, yet it is not correct to assume that these filaments are not approximately parallel at any point in the section. It is also

true that the Price current meter will record the velocity in the direction of its flow whether or not this is at right angles to the measuring section. Any careful hydrographer, however, will make corrections for the general obliquity of the current to the channel cross-section, and for discernible obliquity at any measuring point. Where good channel conditions exist, the obliquity of the water filaments is small, and it must be remembered that it requires an obliquity of about 18° to cause an error of 5 per cent. Such a deviation from the normal position rarely obtains at a good gauging station, and, when this exists, it is usually readily noticeable. Moreover, since the number of points at which obliquity of current occurs, compared with the whole number of points at which measurements are made, is relatively small, a very large error would be necessary in order to produce a considerable error in the total measurement. The inertia of the meter wheel causes it to resist increasing or decreasing its speed of rotation under varying intensities of velocity. The meter, therefore, under rapid pulsations of the water, will not respond fully to either the maximum or minimum velocity, but will record the mean very accurately. The pulsating action of the current causes the Price meter to swing back and forth in the water. When the meter swings up stream it records a velocity higher than that of the moving water, but when it moves down stream it records a velocity lower than that of the water, hence the errors produced by this motion are compensating. Other inaccuracies of a similar nature may exist under certain conditions, but, in a well-rated and properly cared for instrument, these as well as those mentioned above are negligible. Mr. Hanna.

In making flood measurements with the current meter, the necessity of obtaining surface velocities, together with the uncertain relation of surface to mean velocity, make flood measurements by this process inaccurate. Equally great inaccuracies are for several reasons likely to obtain in the use of weirs for determining flood discharge. First, any error in reading the head of water on the weir usually causes a much greater error in the computed discharge than a similar error in gauge reading causes in the meter-measured discharge on account of the greater depth of water at the meter station. Second, the discharge coefficients for weirs are determined with accuracy only for heads of less than 2 ft., and are not determined at all for depths greater than 5 ft. Flood heights have very little respect for these limits, and the use of such coefficients outside of the experimental range is likely to introduce errors as large as those incident to current-meter measurements. Third, unknown leakages around and through the dam may introduce errors of considerable importance. Finally, much discharge measuring is complicated at weirs by flow through water turbines, adding greatly to the uncertainty of the results. The general use of floats for measuring river discharge is im-

Mr. Hanna. practicable, and the resulting data are much less satisfactory than those obtained by any good type of current meter. Therefore, it does not appear that either weir or float discharge measurements can be used satisfactorily as a base to compute the percentage of error in current-meter measurements of river discharge.

Mr. Hoyt. JOHN C. HOYT, ASSOC. M. AM. SOC. C. E. (by letter).—In presenting this paper, the writer's chief object was to point out the non-uniformity in the relation between the two phenomena, rainfall and run-off, and to show that estimates of run-off, based on rainfall as the principal factor, were unsafe, and if used in practice might lead to serious errors. Knowing that this position was not held by some engineers, it was hoped that there would be several discussions by the users of rainfall data. All the participants in the discussion agree in general with the writer except Mr. Vermeule, who claims to be able to estimate stream flow from a knowledge of conditions such as rainfall, evaporation, topography, etc., to within less than 5 per cent. This accuracy the writer believes is as close as average measurements can be made with sharp-crested weirs, which, within their limits of use, are the best known methods—except by calibrated tanks—for measuring stream flow.

Mr. Vermeule does not state of what his estimates consist, whether they were only the mean annual flow, or whether he obtained the monthly and daily distribution of this flow. These latter values are necessary for any complete hydrographic study.

Data in regard to flow during past years are of value in hydraulic studies only to show what may be expected in future years, and the chances of the same conditions of flow repeating themselves in any year are very small. The percentage of difference between the means for even 5 and 10-year periods is quite large.

For example, in Table 10, which gives the flow of the Ohio at Wheeling for 21 years, the difference of the means for the various years and the mean for the 21 years ranges from + 51 to — 28% from the 21-year mean. For the year nearest to the mean, the difference is + 0.9%, and there are 7 years when the difference is less than 8 per cent.

Studying the means in 5-year periods, by taking the first to the fifth years, inclusive, the second to the sixth years, inclusive, the third to the seventh years, inclusive, etc., the percentage of difference ranges from + 17 to — 14% from the 21-year mean, there being 4 years when the difference is less than 5 per cent. Similarly, by 10-year periods, the range of difference is + 3.2 and — 7.4% from the 21-year mean, there being 7 years when the difference is less than 3 per cent. Other long-period records show similar percentages of difference.

Therefore, in making any estimates of future conditions based upon past records of flow, a liberal factor of safety must be used.

TABLE 10.—MEAN RUN-OFF, IN DEPTH, IN INCHES, OF THE OHIO RIVER AT WHEELING, W. VA., 1884 TO 1905, INCLUSIVE, 1, 5, AND 10-YEAR PERIODS.

ONE-YEAR PERIOD.				FIVE-YEAR PERIOD.				TEN-YEAR PERIOD.			
Year.*	Mean run-off, in inches.	Difference from 21-year mean.		Years.	Mean run-off, in inches.	Difference from 21-year mean.		Years.	Mean run-off, in inches.	Difference from 21-year mean.	
		In inches.	Percentage of mean.			In inches.	Percentage of mean.			In inches.	Percentage of mean.
1884-85	17.82	− 4.86	−21	.....	.....	.....	.....	.....	.....	.....	.....
1885-86	21.18	− 1.50	− 6.6	.....	.....	.....	.....	.....	.....	.....	.....
1886-87	22.39	− 0.29	− 1.3	.....	.....	.....	.....	.....	.....	.....	.....
1887-88	17.46	− 5.22	−23	.....	.....	.....	.....	.....	.....	.....	.....
1888-89	24.34	+ 1.66	+ 7.3	1884-89	20.64	−2.04	− 9.0	.....	.....	.....	.....
1889-90	34.20	+11.52	+51	1885-90	23.91	+1.23	+ 5.4	.....	.....	.....	.....
1890-91	30.64	+ 7.96	+35	1886-91	25.81	+3.13	+14	.....	.....	.....	.....
1891-92	22.09	− 0.59	− 2.6	1887-92	25.75	+3.07	+14	.....	.....	.....	.....
1892-93	21.52	− 1.16	− 5.1	1888-93	26.56	+3.88	+17	.....	.....	.....	.....
1893-94	18.03	− 4.65	−21	1889-94	25.30	+2.62	+12	1884-94	22.97	+0.29	+1.3
1894-95	16.29	− 6.39	−28	1890-95	21.71	−0.97	− 4.3	1885-95	22.81	+0.13	+0.6
1895-96	19.71	− 2.97	−13	1891-96	19.53	−3.15	−14	1886-96	22.67	−0.01	−0.04
1896-97	22.59	− 0.09	− 0.4	1892-97	19.63	−3.05	−13	1887-97	22.69	+0.01	+0.04
1897-98	24.61	+ 1.93	+ 8.5	1893-98	20.25	−2.43	−11	1888-98	23.40	+0.72	+3.2
1898-99	22.88	+ 0.20	+ 0.9	1894-99	21.22	−1.46	− 6.4	1889-99	23.25	+0.57	+2.5
1899-00	17.64	− 5.04	−22	1895-00	21.49	−1.19	− 5.2	1890-00	21.60	−1.08	−4.8
1900-01	24.55	+ 1.87	+ 8.2	1896-01	22.45	−0.23	− 1.0	1891-01	20.99	−1.69	−7.5
1901-02	24.66	+ 1.98	+ 8.7	1897-02	22.87	+0.19	+ 0.8	1892-02	21.25	−1.43	−6.3
1902-03	27.09	+ 4.41	+19	1898-03	23.36	+0.58	+ 2.6	1893-03	21.80	−0.88	−3.9
1903-04	25.35	+ 2.67	+12	1899-04	23.86	+1.18	+ 5.2	1894-04	22.55	−0.13	−0.6
1904-05	21.19	− 1.49	− 6.6	1900-05	24.57	+1.89	+ 8.3	1895-05	23.03	+0.35	+1.5
21-year mean	22.68	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
21-year max	34.20	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
21-year min	16.29	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

* The year period is from October to September, inclusive

Mr. Hoyt.

If estimates of less than 10% error are desired, at least 10 years' records should be available. While a 5-year period will probably give data within about 10%, there may be a possibility of a 15% error. In view of the above, it is not clear of what use or value ultra-refined measurements of flow can be in practical hydraulic work.

Considerable discussion has come up in regard to the suggestion of making estimates of flow at points where no measurements have been made, based upon a long series of records in the vicinity of where the data are desired.

The question of estimating stream flow is so broad that it should comprise a paper by itself; therefore, no attempt was made in the paper to enlarge on that subject. In brief, the writer's views on the subject are as follows:

The conditions affecting the flow of streams may be grouped as follows:

Climate.—Rainfall, evaporation, temperature, wind, humidity.

Topography.—Elevation, swamp and lake areas, proximity to large water areas, shape and direction of axis of basin, slope of basin and bed of stream.

Geology.—Dip of strata, porous or compact sub-soil, age of formation.

Vegetation.—Forestry, cultivated land.

Artificial Agencies.—Dams, storage reservoirs, drainage.

These conditions are so inter-related, and so little is known of the laws that govern them, that it is doubtful whether any definite laws in regard to the flow of streams in general can ever be deduced. Tendencies as to what may be expected under certain combinations of conditions can only be considered.

Data, in regard to the flow of a stream, based on anything else than actual measurements at the point in question, are likely to be very largely in error. In fact, it is believed that there is probably no problem with which the engineer has to contend where the possibility of erroneous deductions is as great as in the attempt to estimate the flow of streams at points where no actual measurements have been made.

In case an engineer is in need of accurate data in regard to the flow of a stream at a point where no measurements are available, he should, without loss of time, visit the point in question and establish a gauging station, make a survey of the bed and banks of the stream, and get such local information as may be available in regard to high and low-water conditions. Even though it may be possible to maintain this station for only a short period, it will, it is believed, be a factor of great value in any estimates which may be made.

By comparing these actual measurements with simultaneous records at a near-by long-record station, coefficients can be obtained for

Mr. Hoyt.

reducing the data at the long-record station so that they will apply at the short-record station, and not only can the total flow be thus estimated, but also its probable distribution.

In determining the coefficient to be used, the effects of the various conditions affecting stream flow, mentioned above, should be considered and due allowance made for them; as, for example, if the record at the secondary station is short, the conditions of the probable ground storage should be determined by considering the geology and topography in connection with the rainfall during the months previous to the period of the observations. The relative weight to be given to the actual measurements at the site and the long-time record will depend on the length of each and the conditions during the time of collecting short records, whether extreme or normal.

Tables 11 and 12 give the ratio of run-off at two stations in each of two drainage basins. Although the range for the various individual months is large, the yearly range is quite uniform. Similar tables for other areas show similar ratios. The ratio of discharge as compared with that of drainage areas, however, does not generally agree as closely. This is especially so in case the difference in area is great.

Mr. Merriman has made some especially interesting observations on the accuracy and the methods of the work of the Geological Survey, in that they differ materially from the observations of men who have devoted years to the special study of determining stream flow.

In his criticism, no authority is given in regard to the source of the information from which he has derived his deductions, and the writer is unable to find any record of experiments which bear out his conclusions. In fact, it is difficult to see how anyone who has made any extensive use of the current meter in the collection of stream flow data could make such deductions.

Much has been published* on the question of methods and accuracy in stream-gauging work. These articles give the results of long and varied experience, and show that the methods used by the United States Geological Survey in this work, when executed by trained men, will give results within 5% of those obtainable with sharp-crested weirs. The degree of accuracy in any case will depend on the time and funds expended in making the observations. In regard to the data presented in the paper, it is the writer's opinion that they are accurate to within about 5 to 10%, with the exceptions of the records of the Roanoke at Randolph, the James at Glasgow, and the Appomattox at Mattoax, the records for which may have a possible error of 15 per cent.

* *Transactions*, Am. Soc. C. E. Vol. XLVII; Part B, Vol. LIV; Vol. XII;
Water-Supply and Irrigation Papers, U. S. Geological Survey, Nos. 95-187, and others;
Reports of the Chief of Engineers, U. S. Army;
"River Discharge," Hoyt and Grover.

Mr. Hoyt.

TABLE 11.—MONTHLY DISCHARGE COMPARISONS OF THE SHENANDOAH RIVER AT MILLVILLE, W. VA., AND THE POTOMAC RIVER AT POINT OF ROCKS, MD., GIVING THE MILLVILLE DISCHARGE AS A PERCENTAGE OF THAT AT POINT OF ROCKS.

$$\text{Drainage area ratio} = \frac{3\,000}{9\,650} = 0.31$$

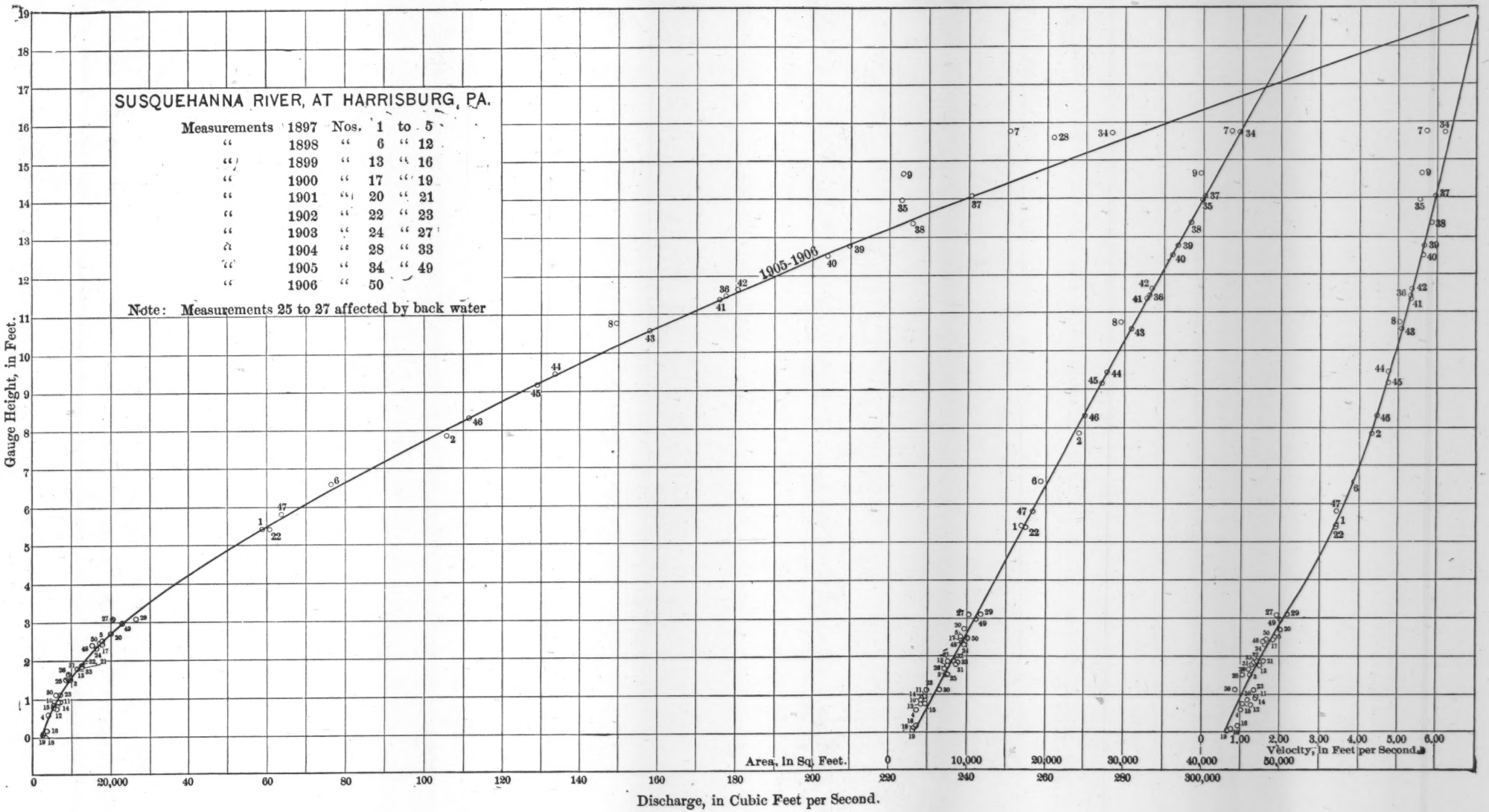
Month.	1898	1900	1901	1902	1903	1905	1906	By months.
Jan.	0.17	0.36	0.37	0.29	0.39	0.24	0.25	0.290
Feb.	0.19	0.29	0.36	0.26	0.25	0.30	0.32	0.266
Mar.	0.14	0.32	0.24	0.26	0.29	0.19	0.24	0.246
Apr.	0.27	0.30	0.32	0.24	0.27	0.30	0.20	0.270
May	0.32	0.38	0.32	0.44	0.37	0.31	0.33	0.338
June.	0.39	0.27	0.44	0.44	0.41	0.39	0.41	0.400
July	0.52	0.46	0.41	0.36	0.27	0.29	0.36	0.348
Aug.	0.47	0.43	0.42	0.43	0.33	0.27	0.34	0.361
Sept.	0.54	0.52	0.33	0.49	0.48	0.25	0.46	0.411
Oct.	0.42	0.71	0.41	0.39	0.45	0.22	0.50	0.446
Nov.	0.33	0.32	0.29	0.41	0.44	0.27	0.48	0.362
Dec.	0.29	0.36	0.32	0.25	0.32	0.22	0.28	0.286
Year.	0.295	0.335	0.34	0.28	0.32	0.255	0.32	0.31

NOTE.—Records incomplete, 1899 and 1904, hence omitted.

TABLE 12.—MONTHLY DISCHARGE COMPARISONS BY RATIOS OF JAMES RIVER, AT BUCHANAN, VA., AND AT CARTERSVILLE, VA., GIVING BUCHANAN DISCHARGES AS PERCENTAGES OF THOSE AT CARTERSVILLE.

$$\text{Drainage area ratio} = \frac{2\,060}{6\,230} = 0.33$$

Month.	1899	1900	1901	1902	1903	1904	1905	By months.
Jan.	0.28	0.27	0.33	0.29	0.30	0.32	0.21	0.29
Feb.	0.33	0.34	0.34	0.42	0.39	0.32	0.16	0.35
Mar.	0.35	0.41	0.47	0.42	0.36	0.39	0.46	0.40
Apr.	0.31	0.25	0.37	0.34	0.33	0.45	0.29	0.34
May	0.45	0.24	0.34	0.21	0.28	0.42	0.42	0.35
June	0.31	0.28	0.34	0.22	0.25	0.47	0.32	0.31
July	0.21	0.28	0.33	0.31	0.33	0.25	0.40	0.34
Aug.	0.12	0.26	0.29	0.22	0.18	0.22	0.17	0.23
Sept.	0.19	0.25	0.24	0.26	0.30	0.21	0.17	0.24
Oct.	0.16	0.43	0.20	0.84	0.24	0.30	0.21	0.21
Nov.	0.11	0.64	0.19	0.19	0.20	0.26	0.23	0.32
Dec.	0.21	0.37	0.48	0.26	0.23	0.17	0.29	0.34
Year	0.31	0.35	0.35	0.31	0.32	0.36	0.32	0.33


SUSQUEHANNA RIVER, AT HARRISBURG, PA.
Measurements 1897 Nos. 1 to 5
" 1898 " 6 " 12
" 1899 " 13 " 16
" 1900 " 17 " 19
" 1901 " 20 " 21
" 1902 " 22 " 23
" 1903 " 24 " 27
" 1904 " 28 " 33
" 1905 " 34 " 49
" 1906 " 50
Note: Measurements 25 to 27 affected by back water
1905-1906
Gauge Height, in Feet.
Area, in Sq. Feet.
Velocity, in Feet per Second.
Discharge, in Cubic Feet per Second.

Mr. Hoyt.

From a study of the literature just mentioned, and from long personal practice, the writer cannot agree that the methods of measuring stream flow are very inaccurate. Measurements of discharge can be made within 5% of results obtained by sharp-crested weirs, and, in fact, the majority of the measurements at the better U. S. Geological Survey stations plat within 3% of a well-defined curve (see Plate LX). The difference is as likely to be plus as minus.

As the measurements at most of these stations extend over a series of several years, and have been made by different persons and instruments, it is evident that there is no radical fault in the methods, for if such existed they would surely appear in such a series of measurements.

Extensive observations show that the larger proportion of the errors in estimates are due, not to inaccuracy in the discharge curves, but rather to inaccuracy in the gauge records. The errors in high-water estimates are due principally to rapid changes in conditions of slope caused by rapid fluctuations in the stage. Consecutive current-meter measurements, covering several days during the rise and fall of a large freshet, show that there may be 10 or 15% difference in the flow at the same stage on a rising and falling stream.

The principal criticism on accuracy by Mr. Merriman is in regard to the small Price current meter. This meter, as used by the Geological Survey, has been developed as the result of the experience of various engineers in the Survey, covering a period of twenty years. The meter has been fully tested, both as to practical working and accuracy. Comparisons between the meter and standard weirs have been made for velocities ranging from ½ ft. to 3 ft. per sec., and show that the two methods agree in general within less than 3 per cent.* The meter ratings show that the meter behaves as well during the high-velocity tests as in low velocities. In the work of the Survey, special care is taken to see that the meters are in good working order, and they are rated sufficiently often to eliminate any errors that might arise from this source.

In making the rating, tests are made with velocities ranging from 0.2 ft. to 10 ft. per sec. Fig. 2 shows a typical current-meter rating curve. In rating the current meter, it is interesting to note that in carrying the meter through still water for a distance of 100 ft. the wheel of the meter will make practically 43 revolutions regardless of the speed with which it is carried over the distance. In other words, it acts in a way similar to that of a wagon wheel passing over the ground. It is noted, further, that the speed may be varied during the run and have practically no effect upon the number of revolutions. If the cups are made to revolve at a very high speed in air, and are then lowered quickly into still water, they come at once to a stop. These

* *Transactions*, Am. Soc. C. E., Vol. XLVII; and Water-Supply Paper No. 95, U. S. Geological Survey.

Mr. Hoyt. facts show that the effect of inertia, mentioned by Mr. Merriman, if any, is very small. The common faults attributed to the meter can usually be traced to the man who is handling it, and it has been found in the work of the United States Geological Survey that the majority of all errors are due wholly to the operator. In brief, it may be said that the experience of specialists, extending over more than a decade, has shown that this instrument will give results with sufficient accuracy for any ordinary hydraulic problem.

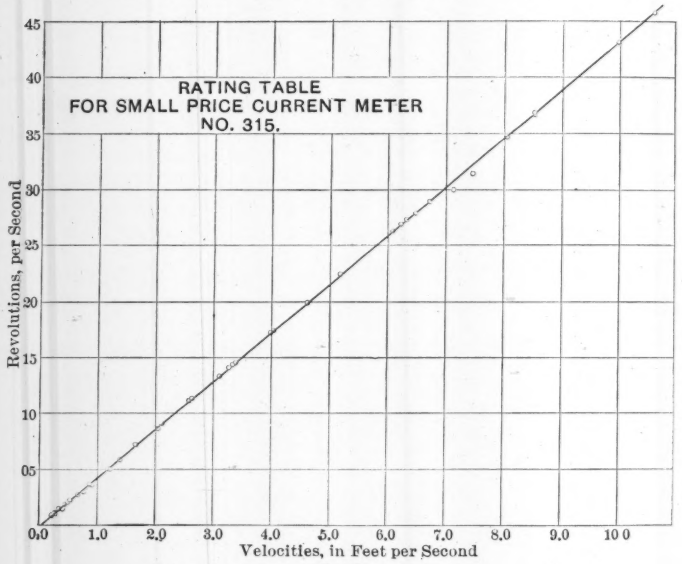


FIG. 2.

Reference has been made to other accurate series of measurements made by other methods. The writer ventures to say that the current meter has been and still is a factor in most series of measurements, and that much of the accuracy in all records can be traced to conditions brought out by the data collected by this instrument. It is believed that if the data and conditions attending the computations of these accurate records were known, they would show uncertainties which might make the results as much in error as those obtained under good conditions with the current meter. Furthermore, the writer submits that, at this time, considering all conditions, there are no better methods known for determining the flow of streams than those used by the United States Geological Survey, and that the accuracy obtainable by these methods, by men who understand their use, is as great as is needed in hydraulic investigation.

MEMOIRS OF DECEASED MEMBERS.

SYLVANUS THAYER ABERT, M. Am. Soc. C. E.*

DIED AUGUST 11TH, 1903.

Sylvanus Thayer Abert was born on July 22d, 1828. He was a son of the late John James Abert,† Hon. M. Am. Soc. C. E., Chief of Topographical Engineers, United States Army, and great-grandson of Colonel Timothy Matlack, whose Quaker ancestry settled in New Jersey in 1677. Colonel Matlack served during the Revolutionary War as Colonel of Militia from 1775 to 1777; Deputy to the State Conference in 1776; Delegate to the Continental Congress in 1780-81; and as Secretary to the Council of State, and Master of the Rolls, in 1781. He died in Philadelphia in 1829.

Mr. Abert was graduated from Princeton College in 1848, and, as a rodman, he commenced his first engineering work upon the extension of the James River and the Kanawha Canal, then terminating at Lynchburg, Virginia. This canal was designed to connect tidewater at Richmond, Virginia, with the Ohio River, by way of the Kanawha, but was never completed beyond Buchanan, Virginia. Between Richmond and Lynchburg it formed the main line of travel and traffic. The only railroad in Virginia at that time connected Richmond with Fredericksburg, with branches to Acquia Creek and Quantico. The route thence to Washington was by steamer on the Potomac. An account—an amusing travesty—of a journey, made in 1842, from Potomac Creek to Fredericksburg may be found in Dickens' "Notes on America."

In 1849 Mr. Abert was called to Washington to make a survey of the Potomac between the aqueduct over the river at Georgetown and the Long Bridge. He prepared a chart of the river, and recommended the removal of parts of the bridge and the substitution of longer spans.

In 1850 he surveyed the canal which enters the Eastern Branch near the Arsenal, and recommended connecting it with the Chesapeake and Ohio Canal, then extending into Washington on the north side of the Smithsonian grounds.

He made a survey of Cape Vincent Harbor, New York, in 1852, and recommended a breakwater. Two years later he surveyed Tar River, North Carolina, and the construction of the lock and dam there was commenced. In 1855 he made a survey of Beaufort Harbor, North Carolina.

Mr. Abert's interesting report of the survey of New River Inlet, about 40 miles northeast of Wilmington, North Carolina, in 1856, at-

* Prepared by the Secretary from notes furnished by Mr. Abert in 1897.

† A memoir of John James Abert, Hon. M. Am. Soc. C. E., is printed in *Proceedings*, Am. Soc. C. E., Vol. XIX, p. 88.

tracted the attention of Professor A. D. Bache, of the United States Coast Survey. The formation at the mouth of New River received careful study. The position of the delta, built within the coast line by storms, is the reverse of the delta of the Mississippi, which, lying without and beyond the coast line, extends its channels to the Gulf of Mexico, like the outstretched fingers of the hand. Formed by silt brought down by the Mississippi, it diverges seaward; the delta of New River, formed by sand brought in by the ocean storms, diverges landward; but, whether extending landward or seaward, each delta exhibits the same anastomizing character, in obedience to the dominating force. In New River the storm force dominates the ebb and the fluvial. The hindrance of the ebb was made apparent by conditions observed during the survey. The tide, passing up the channel, and being detained in the bay above, left an unexpended head of water, where the flood again set inward. To improve the channel, a new relation of forces was necessary, in order to increase the volume of outflowing water; and, to attain this, Mr. Abert proposed deepening and widening the channel, following an easy curvature; he also proposed the fixation of the banks by the use of stones and fascines.

The value of the sounds of Carolina as channels for interior navigation has long attracted the attention of engineers. Part of Mr. Abert's report was devoted to the importance of improving these sounds for coastwise trade and for the transportation of heavy guns and material of war. With the exception of a distance of about 244 nautical miles (between Beaufort, North Carolina, and Charleston, South Carolina), interior navigation, without interruption, extends from Norfolk to Fernandina, a distance of about 1 745 nautical miles. A Government torpedo-boat has safely made the passage through inland channels and by way of Romney Marshes, from Charleston to Fernandina, and a comparatively small amount of excavation would make this channel navigable for war vessels of the same size as the *Amphitrite*. Between Appalachicola and Lake Pontchartrain, by connecting the deep sounds along the coast, the route could be continued to New Orleans.

Northward from Beaufort, North Carolina, an uninterrupted channel exists, by way of bays and canals, as far as New York, and thence by way of the Erie Canal to Lake Erie. The Mississippi can then be reached *via* Chicago, by way of the Illinois and Michigan Canal, thus completing the connection between the Sounds of Carolina and the Mississippi River.

Mr. Abert's report was published in *The Wilmington Journal* (Wilmington, North Carolina), on April 23d, 1856. It may be interesting to note that at this period the Southern States conducted public improvements within their borders, and many of them appointed a State Engineer to direct all public works.

In 1857 Mr. Abert was employed as Assistant Engineer on the construction of Fort Delaware, on Pea Patch Island, in the Delaware River, about 40 miles below Philadelphia. This was a casemated work, founded on piles and surrounded by a wet-ditch. Its embrasures were provided with wrought-iron shutters, and it was believed to be impregnable to the artillery of that day.

In 1858 Mr. Abert was ordered to the Warrenton Navy Yard, Florida, and placed in charge of the buildings, works, and ground within the Naval Reservation. The works within the reservation—about 7 miles from Pensacola—were varied and interesting. Mr. Abert built a foundry for heavy castings; constructed 200 ft. of sea wall in 30 ft. of water, by using a diving bell; built a machine shop for the repair of ships; submitted plans for a ship house and launching ship; and prepared plans for a ship basin and graving dock, which were approved by a Board of Engineers appointed by the Secretary of the Navy. The foundation of the dock presented especial difficulties. The yard occupies a tract of sandy soil about 3 ft. above the level of the bay. Borings revealed a stratum of clay 40 ft. below the surface, and another stratum was found about 40 ft. lower. It was evident that the dock would have to be built in water. The plan proposed was as follows:

After dredging the entire area, piles were to be driven at not less than 3 ft. from center to center and then sawed off. For the bottom and side walls, concrete was to be deposited from staging—largely floating staging. After closing the outer end of this concrete box temporarily, the bottom and sides were to be finished with heavy granite facing. A wrought-iron floating gate was designed to close the dock when in use. These plans are now in the Bureau of Yards and Docks at Washington.

Mr. Abert was interested in the Bay of Pensacola, and wrote an official letter recommending Pensacola Harbor as the site of a National dockyard and arsenal. A fleet can find anchorage in its waters, and there are 22½ ft. of water on its bar at mean low water. It is connected by rail with Montgomery, Alabama, and the interior of the country, and the Gulf of Mexico does not contain a better harbor.

There was a general suspension of public improvement works at the beginning of the Civil War, and Mr. Abert reported for duty to the Chief Engineer of Bank's Division, at Frederick City, Maryland, in December, 1861. His services in the campaign in the Valley of the Shenandoah are mentioned in the "Personal Recollections of the War," by General David H. Strother (Porte Crayon), published in *Harper's Monthly*, in March, 1867.

Mr. Abert established a flying bridge at Williamsport, Maryland, upon which the retreating army passed over the Potomac. In 1862 he was at Warrenton Junction, Virginia. He was in Nashville

when Hood was defeated, and built Fort Garasche, north of the city and overlooking the Cumberland. He served in the campaign before Petersburg, and was with Meade's Army, under orders of Major Michler, of Meade's engineer staff, until the surrender at Appomattox on April 9th, 1865.*

In 1865 and 1866 Mr. Abert was one of a party of engineers called by the United States of Colombia to make surveys of certain localities and to prepare plans of improvement. The party left New York on December 11th, 1865, for Aspinwall, and, after a short visit to Panama, they returned to Aspinwall and embarked in an English steamer for Santa Martha. At this point work began on December 29th.

The surveys and plans of improvement were made for the following localities:

First.—For connecting by rail the Town of Santa Martha with the Town of Cienega, on the delta of the Magdalena.

Second.—For improving the channel through *canos* of the delta, between Cienega and Barranquilla.

Third.—To survey "the Canal de la Pina and *cienigas* (lakes) and *canos* (channels) through which a navigable route can be made from Barranquilla, on the Magdalena, to Sabanilla, the seaport at the mouth." This canal was "to be of such dimensions as to admit at all seasons the passage of the largest steamers navigating the Magdalena."

Fourth.—The next work was at the Island of Margarita, on the Magdalena, about 140 miles above Barranquilla. This island, of great beauty and fertility, is not less than 40 miles in length. The course of the river had been diverted by a freshet from the right side of the island where it flowed past the flourishing city of Monpox—noted for its "dulces," and famous for its ornaments of gold-wire work—to the left side where it passed the site of the insignificant town of Manganguy. Here were held annual fairs, frequented by merchants of Barranquilla, who readily disposed of all their wares to the natives from the surrounding country. Many of these merchants were natives of Bremen and Hamburg. The Monposians asked the engineers to convert the river to its orthodox course. All that was needed for this object was a jetty.

Fifth.—A reconnaissance was made and a map prepared of the Magdalena from Honda (head of navigation) to Barranquilla, a distance of about 525 miles.

The party returned to New York during the latter part of May, 1866. A revolution preceded the arrival of the engineers in Colombia, another followed soon after their return—the chronic condition of this interesting country.

* Mr. Abert's services are mentioned in somewhat more detail in the statement prepared by him for the Society of the Army of the Potomac.

In 1867 Mr. Abert was in charge of repairs at Fort Foote, on the Maryland side of the Potomac, about a mile below Alexandria.

Mr. Abert's next duty (1869) brought up the question of the connection of the Great Lakes with the Mississippi. He was charged with the survey of the Illinois River from La Salle to its mouth at Grafton. The collection of data—of little interest to the general reader—is an essential object in every survey; the following facts are taken from Mr. Abert's report:

The distance from La Salle to Chicago, by way of the river and the Illinois and Michigan Canal, is	99	miles.
From La Salle to Grafton is.....	223	"
From Chicago to Grafton is.....	322	"

The Illinois and Michigan Canal connects Chicago with the Illinois River. The average fall of the river below La Salle is 0.15 ft. per mile, and the entire fall is 29.6 ft.

Mr. Abert submitted an estimate for an improvement by dredging, and also by locks and dams, and pointed out the importance of this route as a navigable connection between the Lakes and the Mississippi. In his report he says:

"Its real importance can be estimated by regarding it as completing a system of water communication between the East and the West, of which the Erie Canal constitutes an important part."

Under the heading "Lake Michigan as a Reservoir," he considers the importance to navigation of the lake as a feeder to the Illinois at low water. The report states:

"The close of the season did not permit a special examination or survey with a view to the complete solution of this question. Facts and theoretical deductions are herein presented, sufficient to indicate the course of examination to be pursued, in order to solve the problem."

Taking the discharge at Hennepin as a basis, and a width of channel of 350 ft., with a velocity of a little more than one mile per hour, a depth of not less than 8 ft. could be given on all bars during the low-water period, which prevails from 60 to 90 days.*

This project was not approved, and in 1868 was declared "impracticable at any reasonable cost."†

The City of Chicago, unaided by Government, has solved the problem of constructing a ship canal. Mr. Abert wrote, in 1897:

"The year 1898 will see the completion of this stupendous work, which will turn the waters of the Great Lakes into the Mississippi."

* Ex. Doc. 16, 40th Cong., 1st Sess., House of Reps. Also Reports of Chief of Engrs., 1868, pp. 445-449.

† Ex. Doc. 116, 40th Cong., 2d Sess., House of Reps.

A writer observes:

"The solution of the problem of disposing of the city sewage has rendered practicable one of the boldest schemes of improvement of internal navigation which has ever been entertained by engineers. When completed, this canal will have a width of 162 ft., a depth of 25 ft., and a length of about 35 miles. Its maximum flow will be about 600 000 cu. yd. per min., and will increase the Illinois eighteen times in the upper part, twelve times at the mouth at low water, and about 25% at St. Louis, as estimated. For details of this remarkable work the reader is referred to the reports of the engineer, in which he will find an account of the machinery used, the channelers, drills, hoisters, cantilever lifts, and dredges, and statements of the value, kind, and quantity of the work done by them. Thirty millions of dollars will be expended before the final completion of the work, if anything can be called final to a people so energetic and liberal as the citizens of Chicago."

In 1869 Mr. Abert was charged with the survey of the Arkansas River, from Fort Gibson, on the Neosho, in Indian Territory, to its mouth, in the Mississippi. The party was recruited in Cincinnati and started in January, 1869. The steam dredge *Octavia* towed the survey boat to Memphis; thence it was towed to Little Rock by a river steamer, and another steamer conveyed it to Fort Gibson, which was reached on February 7th. On the morning after arrival, one of the party, who had been ailing for some days, was pronounced by the surgeon of Fort Gibson to have confluent smallpox. There were no railroads in Arkansas at that time. Without medical aid, and accompanied by threats of the Indians, the survey began. Five sick men, quarantined in camp, were transported from place to place as the survey advanced. One man died, and was buried on the bank of the river.

The following data are gathered from the report: The distance from Fort Gibson to Little Rock is 289½ miles; the fall is about 242½ ft. Discharge measurements, about 30 miles apart, were taken according to the method of Humphreys and Abbot. Careful record was made of the geology and products of the valley. High water prevailing between January and July, no low-water survey of the bars was practicable. For a channel so variable in its course, and so changeable in the depth and position of its bars, any low-water survey would have been of small value until certain important and preliminary work had been completed. Large tracts of river front are washed away during every freshet, and acres of valuable cotton land are annually destroyed by the current. Therefore, no satisfactory estimate for the improvement of the bars can be made until the banks have been fixed. A small amount of work was recommended at but four localities.

The general conclusion stated in the report was:

"That the first object of improvement should be to give permanency to the banks by protecting them from the abrasion of the current."

The regimen of the river would be established by this means, and the alveus of the stream could then be permanently improved.

Maps and diagrams accompany the report, and profiles and cross-sections, of velocities and oscillations, relating to the hydraulics of the river, are fully represented. The survey* was completed in July, 1869.

In 1871 Mr. Abert was engaged on the survey of the Cumberland River, in Kentucky and Tennessee, and the following facts are gleaned from his report:

From the Great Falls, in Pulaski County, Kentucky, to its mouth, the river is 595 miles long. The Great Falls have a vertical descent of 56 ft. The scenery, as far as Point Burnside, is wild and picturesque. At the rapids the rocks rise in vertical escarpments several hundred feet, and the water flows in a thin sheet over the horizontal stratum of limestone, left bare at low water. Here is a geologist's paradise, for myriads of encrinites are entombed in the rock. From Point Burnside to Nashville the distance is 326½ miles, and the descent 232 ft. at low water. From Nashville to the mouth the distance is 92 miles, and the descent 79 ft.—where the river enters the Ohio, about 73 miles above Cairo.

The obstructions to navigation were:

1.—Overhanging trees;
2.—Ledges of rock;
3.—Bars composed of the debris of bluffs;
4.—Bars of gravel and ferruginous sand;
5.—Obstructions in the lower river, such as sunken wrecks—the remains of the war.

Below Point Burnside the method of improvement consisted chiefly in contracting the stream at designated points.

Under the heading, "Resources" the report gives a brief description of the geology and a short history of the coal mines of Pulaski County and of Poplar Mountain. The first mines are above Point Burnside; the Poplar Mountain mine is 58 miles lower down and about 268 miles above Nashville.

The Tennessee Rolling Mills, below Nashville, are at the point where the river passes again into Kentucky. A fine quality of boiler iron is made from the brown hematite found buried on the banks of the river in the form of geodes from 1 to 2 ft. in diameter. These geodes often contain fossil crinoids and crystals of lime. The plates made from this ore are used extensively in the manufacture of the boilers of boats on western waters.

The advantage of a ship canal across the Isthmus connecting Central and South America has long attracted the attention of scientists

* Ex. Doc. 295, 41st Cong., 2d Session.

and engineers. Humboldt devotes several pages to its consideration. Hughes' favorable opinion called public attention to Panama, and Strain's expedition across Darien was led to misfortune by the mythical account of Gisborne. Michler's able survey of Atrato, and Childs' clever report on Nicaragua gave hope of a feasible route; but all these surveys and reports leave the mind in doubt as to which route should be selected for a ship canal. Each possessed advantages and disadvantages. Upon some lines, harbors on the Pacific were separated from harbors on the Gulf by an insurmountable barrier, or one that could only be penetrated by a costly tunnel. On other lines, while the divide opposed no difficult obstruction, harbors are wanting at the termini. To form an intelligent opinion, Mr. Abert attempted to bring together all the information to be found in reports relating to this subject. While in Cincinnati, in 1872, he published a pamphlet having the following title: "Notes, Historical and Statistical, upon the Projected Routes for an Interoceanic Ship Canal between the Atlantic and Pacific Oceans, in which is included a short Account of the Character and Influence of the Canal of Suez, and the Probable Effects upon the Commerce of the World, of the Two Canals, Regarded either as Rivals, or as Parts of one System of Interoceanic Travel."

This pamphlet* was given a wide publication. It was printed by Murat Halsted in a broadside of *The Cincinnati Commercial.* It was favorably commented on by R. J. Walker, ex-Secretary of the Treasury, by W. H. Seward, Secretary of State; by Nathan Appleton; by the editor of the *Journal* of the Franklin Institute; and by *The Nation,* of New York. Though valuable at the time of publication, its information has been superseded by later surveys, and by the work done on the Nicaragua and Panama Canals; but a navigable way across the Isthmus is still an unsolved problem.

In 1873 Mr. Abert was called to important duty upon the Atlantic seaboard. By order of the Secretary of War, he was placed in charge of a geographical division beginning at Washington, D. C., and terminating at Wilmington, North Carolina. His annual reports relate to the surveys and improvements of rivers and harbors in Maryland, Virginia, and North Carolina. Among the more important are those relating to Norfolk Harbor, Elizabeth, York, Rappahannock, and Roanoke Rivers, the reclamation of the flats at Washington, and the survey of an inland line of navigation between Norfolk and Wilmington.

The plan for the reclamation of the Potomac flats was submitted to a Board of Engineers, appointed by General A. A. Humphreys. The problem was complicated with the sanitary conditions, caused by a large flat extending in front of the city. Beginning with a width of ½ mile at Georgetown, the river widens to 1 mile at the Long Bridge,

* Published by R. W. Carroll and Company, Cincinnati, 1872.

about 3 miles lower down, and the flats were traversed by two channels (in 1790 by three channels). It was proposed to reclaim this area, near the Washington side, by a deposit raised above the level of the highest water, and conforming in outline to the hydrographic curves. The area covered by the reclamation was about 700 acres—an addition to the city largely exceeding in value the money to be expended upon it.

The canal to connect the waters of Norfolk Harbor with the Cape Fear River at Wilmington is so important that it deserves some notice. It may be described briefly under three heads:

1.—An inland navigation, near the coast, and chiefly by way of the sounds;

2.—Physical features of the coast;

3.—Historical notes and old maps.

In relation to the canal, estimates are submitted, but Mr. Abert recommends that nothing be done until further examinations have been made. Other routes are mentioned, and an examination of each is regarded as necessary before the problem of location can be solved. Description is given of the Kearney route, and the report of Colonel Kearney is printed in the appendix. Under the headings, "Physical Description" and "Coast Line," quotations are made from the following maps: Hariot's, 1585; Lauson, 1708; Wimble, 1738; Mouzin, 1775; Atlantic Neptune, 1780; Lewis, 1795; United States Coast Survey, 1875. Copies of these maps accompany the report. A study of them exhibits interesting changes in the coast line. Attention is called to the progressive closing of the inlets, and to the southward and southwestward movement of the sand drives.

Under the heading "Changes of the Inlets," comparison is made of the condition at the time of survey and the condition represented on the map of Hariot (1584). Comparison is also made with the statement of Colonel William Byrd, Commissioner on the part of Virginia and North Carolina. Colonel Byrd's observations, made in 1713, are contained in the famous Westover manuscript. The first accurate mapping of the coast line will be found in the charts of the Coast Survey. With this aid we can trace the rate of "Hatteras' advancing foot," and measure the retreat of the coast line between New River Inlet and Masonborough, an inlet near Wilmington.

Five expeditions were sent out by Sir Walter Raleigh, between 1584 and 1590, to the coast of Virginia—now North Carolina. Extracts are given from voyages and travels, published by De Bry Brothers, in Frankfort in 1590, and from Hakluyt's Collection "reprinted in London in 1600." These extracts are accompanied by comments in which Mr. Abert endeavors to follow the steps of the exploring party after landing at Roanoke Island. No permanent settlement

was made by the five expeditions sent by Sir Walter to the coast of Virginia, but the writers mentioned supply valuable matter to one who wishes to study the changes in the coast.

Included in the appendix is an interesting letter from Colonel Marshall Parks, President of the Albemarle and Chesapeake Canal, also one from Mr. McAlpin C. Engs. Tables of statistics of trade are added. If not the most interesting, this report includes much valuable matter. Other reports by Mr. Abert, on the rivers and harbors of Virginia, Maryland, and North Carolina, will be found in the Reports of the Chief of Engineers between the years 1873 and 1890.

Mr. Abert was elected a Member of the American Society of Civil Engineers on September 21st, 1870.

CHARLES BLACKWELL, M. Am. Soc. C. E.*

DIED DECEMBER 29TH, 1906.

Charles Blackwell was born at Devizes, Wiltshire, England, on February 4th, 1843. He was the eldest son of Thomas Evans Blackwell, the first Vice-President and General Manager of the Grand Trunk Railway System, and came to Canada with his father in 1857. He was descended from a family of engineers, his grandfather, Thomas Blackwell, having built several stone bridges over the river Avon near his native town of Devizes, Wiltshire. His father, Thomas Evans Blackwell, was the first Hydraulic Engineer in England, and built the Kennet and Avon Canal. Mr. Blackwell attended the High School and McGill College at Montreal.

In 1860 he entered the service of the Grand Trunk Railway Company, where he remained until 1869, being employed principally in the Motive Power Department. From 1869 to 1876 he was engaged upon the location and construction of the Intercolonial Railway, having charge of a 22-mile section for three years, and of a 74-mile section for two years of this time. During this latter period he had supervision of the equipment and of the erection of the machinery in the principal repair shops at Moncton, New Brunswick.

During 1876 and 1877 he was Resident Engineer in charge of the completion of 45 miles of the Quebec, Montreal, Ottawa and Occidental Railway. From 1877 to 1879 he had charge, as Resident Engineer, of the improvement and construction of the Quebec Central Railway. In 1879 he was appointed Mechanical Engineer of the Dominion Department of Railways and Canals, and had prepared, under his direction, detailed specifications and drawings for the construction of all kinds of rolling stock required for the Canadian Pacific Railway, and also specifications and drawings for the water service on that road.

Upon the completion of construction on the Canadian Pacific Railway, in 1881, and the turning over of the line to the operating company, Mr. Blackwell went to Roanoke, Virginia, as Superintendent of Motive Power of the Norfolk and Western and the Shenandoah Valley Railroads. The Roanoke shops were constructed and equipped according to the plans and specifications prepared by him. The building and equipping of these shops occurred at the time when modern shop practice was just developing, when heavier engines and cars were just arriving, and when the locomotive and car equipment of it was only dimly foreshadowed, but dreamed of; and yet the Roanoke shops, while they have been extended, remain—as far as their particular buildings are concerned—as Mr. Blackwell planned them, and are

* Memoir prepared by Walter S. Newhall, Esq.

well known for their capacity and economical efficiency under present conditions.

Mr. Blackwell left the Norfolk and Western Railroad in 1885, and was engaged on the Union Pacific Railroad for three years, during one year of which he was Manager of the Montana Division.

In 1888 he went to the Central of Georgia Railway as Engineer of the Machinery Department, and remained with that company at Savannah, Georgia, for two years. On account of ill-health he had to come north again, and from 1891 to 1897 was not engaged in railway work; four years of this time he was with the Schoenberger Steel Company, his headquarters being in Pittsburg.

He again entered railroad service at Toledo in 1897 as a special Mechanical and Civil Engineer in the office of the Receivers of the Wheeling and Lake Erie Railroad, remaining with this company after its reorganization, and becoming Principal Assistant Engineer subsequent to August, 1900. During the time he was engaged on the Wheeling and Lake Erie Railroad he made many investigations and reports on that and other properties. In 1903 Mr. Blackwell was tendered the position of Special Assistant to the Chief Engineer of the Wabash Railroad, and entered the service of that company on January 1st, 1904, where he remained for two years.

During the last six years of Mr. Blackwell's life the writer of this memoir was fortunate in knowing him intimately, both socially and professionally, and formed a strong attachment for him. Accurate, methodical, resourceful, with a keen mind for detail and a fund of information acquired by long and varied experience, both as a Mechanical and Civil Engineer, he was especially well adapted to fill the confidential positions which he occupied during the last eight or ten years of his life.

Loyal, kind, and generous, a student of the times, both past and present, he was a charming companion. Born a gentleman, he remained one to the end.

During the last year of his life his health failed rapidly, and he died of Bright's disease, at Cincinnati, on December 29th, 1906.

On December 27th, 1878, Mr. Blackwell was married to Emily, only daughter of Edward B. Chandler, Esq., Barrister, of The Grange, Moncton, New Brunswick, and granddaughter of The Honorable Edward Barron Chandler, Commissioner of Railways, and Governor of the Province of New Brunswick. Mrs. Blackwell and three sons, Thomas Everette, Hubert Charles, and John Buckland, survive him.

Charles Blackwell was elected a Member of the American Society of Civil Engineers on September 7th, 1881. He was also an Associate Member of The Institution of Civil Engineers of Great Britain; an Honorary Member of the American Railway Master Mechanics Association; and a Member of the Master Car Builders' Association.

LOUIS GUSTAVE FREDERIC BOUSCAREN, M. Am. Soc. C. E.*

DIED NOVEMBER 6TH, 1904.

Louis Gustave Frederic Bouscaren was born on the Island of Guadeloupe, on August 26th, 1840. Both his parents were of old West Indian families of French descent, his great-grandfather having been awarded a large part of the Island of Guadeloupe, with the title of Marquis, by the French Government for distinguished services rendered by him as a member of the French Military Engineer Corps.

In 1850 the parents of Mr. Bouscaren, who were quite wealthy, came to the United States, settling on a large tract of land which they had purchased in Grant County, Kentucky, a part of which, with the old homestead, is still retained by the family.

Mr. Bouscaren, who was the oldest son of eight children, received his earlier education at home, but at the age of twelve he was sent to St. Xavier's College at Cincinnati, Ohio. Two years later he was sent to France, where he entered the Lycée St. Louis, one of the great Government schools in Paris, remaining there until he was twenty, after which he entered L'Ecole Centrale des Arts et Manufactures, from which he was graduated with high honors, in 1863, as "Ingénieur des Arts et Manufactures."

His first employment, after his return, was as Draftsman in the office of S. Hannaford and Sons, leading architects in Cincinnati, and, later, while awaiting an opportunity to become connected with outside engineering work, with the firm of Lane and Bodley, manufacturers of mill machinery and engines.

In 1865 Mr. Bouscaren was appointed Assistant Engineer of Maintenance-of-Way on the Ohio and Mississippi Railroad, under the late Thomas D. Lovett, M. Am. Soc. C. E., Chief Engineer, and later under the late E. C. Rice, M. Am. Soc. C. E., who succeeded Mr. Lovett. He served under these two engineers in various positions for the next 11 or 12 years In 1868-69, as Assistant to Mr. Rice, he made the preliminary and location surveys, and later was in charge of construction of the St. Louis, Vandalia and Terre Haute Railroad. From 1869 to 1872 he served as Principal Assistant to Mr. Rice on the St. Louis and South Eastern Railroad, at the same time acting as Chief Engineer of the O'Fallon and Belleville Railroad. In 1872-73 he was Principal Assistant on the Cairo and Vincennes Railroad. In these several positions Mr. Bouscaren showed a marked faculty for details, which, together with his close application and skill, led to his being entrusted with greater responsibilities.

In 1873, Mr. Lovett, then Principal Engineer of the Cincinnati Southern Railway, engaged him to prepare the specifications for, and

* Memoir prepared by G. H. Benzenberg, President, Am. Soc. C. E.

to take charge of the construction of, the great railway bridge for this road over the Ohio River at Cincinnati. Immediately following this he was appointed First Assistant Engineer, Department of Construction, of the Cincinnati Southern Railway, which position he held until late in 1876, when, upon the resignation of Mr. Lovett, he succeeded him as Consulting and Principal Engineer, and completed the construction of the road.

In the preparation of the plans and specifications for this bridge, which was to embody, at that time, the longest truss span in the world, Mr. Bouscaren entered upon a comprehensive series of tension and compression tests of full-sized bridge members and worked out new formulas based upon the results of these tests. He also introduced in his specifications new forms for considering concentrated loads, and in many ways advanced the requirements for bridge construction, especially in the matter of details, so that his specifications for the many bridges which he, as Principal Assistant and later as Chief Engineer, built for the Cincinnati Southern, were accepted as models by many engineers.

Among many bridges, the noted Kentucky River Bridge may be mentioned as the first great cantilever bridge in this country. This bridge was built by the late C. Shaler Smith, M. Am. Soc. C. E., and was first designed to be a continuous bridge, but Mr. Bouscaren changed this design, fixing the points of contraflexure by cutting the lower chords, enclosing the ends in loose sleeves.

The line of the Cincinnati Southern Railway runs through very hilly and mountainous country, and the many difficulties natural to such a location were successfully overcome by Mr. Bouscaren's engineering skill. The construction included some twenty-seven tunnels, the most important of which was the King's Mountain Tunnel, more than 4 000 ft. long.

In 1881, when the Cincinnati Southern Railway was leased by the Erlanger Syndicate, Mr. Bouscaren was made Consulting and Chief Engineer of the entire railroad system controlled and operated by this Syndicate. In this capacity he constructed the New Orleans and North Eastern Railroad, and completed the construction of the Vicksburg, Shreveport and Pacific Railroad. He also did a large amount of reconstruction work on the Alabama Great Southern and on the Vicksburg and Meridian Railroad. In connection with the New Orleans and North Eastern Railroad, he constructed a pile trestle, 21 miles in length, where the railroad crosses the Lake Pontchartrain marshes. All the timber used in this trestle was creosoted, for which purpose he designed and had built the most extensive creosoting plant in the country at that time.

In 1886 Mr. Bouscaren resigned his position as Chief Engineer, and opened an office in Cincinnati as Consulting Engineer. The repu-

tation which he had established as a careful, capable and conscientious engineer secured for him a considerable amount and variety of work. He was engaged in the construction of a number of bridges across the Ohio River, among them the Central Bridge between Cincinnati and Newport, which he designed, and the Chesapeake and Ohio Railroad Bridge. He also had charge of the strengthening of the cables of the suspension bridge between Cincinnati and Covington, and designed and constructed the Fourth Street Bridge over the Licking River in Covington, Ky. He was a member of the Commission appointed by President Cleveland to examine into and report upon the best plan to bridge the North River.

From 1887 to 1891 Mr. Bouscaren was Chief Engineer for, and built, the Covington, Ky., Water-works; he also built the Norwood Water-works. In July, 1896, the Board of Trustees, "Commissioners of Water-works," appointed him Chief Engineer of the new works which were to be constructed for the City of Cincinnati, in which work Charles Hermany, Past-President, Am. Soc. C. E., and the writer were associated with him as Consulting Engineers. To this work he devoted his entire time and his best skill and energy up to the day of his death, and so far as the same was planned by him, it will stand as a lasting monument of his ability and eminence as an engineer.

Into all of his work Mr. Bouscaren introduced an individuality based upon his independence of research and conclusion, which very often led him in advance of contemporaneous practice. He was a close student and a constant reader and had acquired a very large library of scientific and professional works. His plans and specifications were always clear, comprehensive and explicit, and he would never allow a contractor to swerve or deviate in the least from a full compliance with every obligation of the contract and specifications. He was firm, strict and insistent, yet at no time unjust and unfair. Honesty of purpose and integrity were leading characteristics in all his business transactions.

Socially Mr. Bouscaren was of a delightful and generous disposition, always unselfish and unassuming, and ever courteous in his demeanor toward his assistants and co-laborers. Being a devout member of his Church, he was also liberal in aiding her charities.

On April 18th, 1876, Mr. Bouscaren married Miss Helen Seymor Lincoln, daughter of Timothy D. Lincoln, a distinguished lawyer of Cincinnati. Twenty-five years of happy home life followed this union, which grew to a family circle of three sons and two daughters. Mrs. Bouscaren's death, in July, 1901, was a severe blow from which he never fully recovered, especially as his children shortly after left their home to attend school, the oldest son being at the Massachusetts Institute of Technology, the second son studying law at Yale, while the daughters attended school in the Convent at Washington, D. C.

Shortly after attending the Annual Meeting of the American Society of Civil Engineers, in January, 1903, Mr. Bouscaren was attacked with severe pains and internal hemorrhages which compelled him to avoid much walking or exercise, and which necessitated his remaining much of the time either at his home or at the hospital, where he underwent two painful operations—which gave him little relief—followed by long periods of suffering, during which he displayed wonderful patience and fortitude, regretting only that he could not oversee and finish the work he had undertaken. Despite all the efforts of eminent surgeons and physicians, he became so weak that on November 6th, 1904, surrounded by his children, his brother and sisters, he quietly passed away.

Thus ended an active, sincere and useful life, full of good deeds and creditable work, whose record is that of an honest, upright, capable and eminent engineer.

On April 7th, 1875, Mr. Bouscaren was elected a Member of the American Society of Civil Engineers. He served as a Member of the Board of Direction in 1881; and held the office of Vice-President at the time of his death. He also served on a number of special and other committees, and was a frequent contributor to the professional papers and discussions.

In June, 1880, he was elected a Member of the Société des Ingénieurs Civils de France, and, on December 7th, a Member of the Institution of Civil Engineers of Great Britain.

JOHN EUGENE CHENEY, M. Am. Soc. C. E.*

DIED SEPTEMBER 25TH, 1906.

John Eugene Cheney was born in Lowell, Massachusetts, on February 12th, 1847. His parents were John L. Cheney and Cynthia Cram. His father was a direct descendant of Hannah Dustin. His parents lived originally in Ware, New Hampshire, from which town they moved to Lowell. In that city his father was Superintendent of the Merrimack Mills for many years, and afterward was engaged in manufacturing spools, bobbins, and shuttles.

In 1865 Mr. Cheney was graduated from the Lowell High School, and at once entered the Lawrence Scientific School of Harvard University. He was obliged to leave, however, after completing one year of the two-year course, but, in 1900, in recognition of the valuable professional work which he had done, the University conferred upon him the degree of S. B. as of the Class of 1867. This degree was prized more highly by Mr. Cheney than if it had been obtained in the usual way.

Mr. Cheney's first work, after leaving the Lawrence Scientific School, was at the Charlestown Navy Yard. He was also for a time with E. D. Leavitt, Jr., M. Am. Soc. C. E. He entered the employ of the Louisville Bridge and Iron Works, Louisville, Ky., in 1870, and remained there four years, obtaining valuable experience which served as the foundation of his future eminence as a structural engineer. He became an Assistant Engineer in the service of the City of Boston in 1874, and was appointed Assistant City Engineer in 1885. This position he held until his death, having acted frequently as City Engineer in the absence of the latter. During this period he had charge of the city bridges, and his work was of great value to the city, as he proved himself to be a very conscientious, capable and thorough structural engineer.

In Mr. Cheney's thirty-two years of service nearly all the city bridges were built or renewed under his supervision. The bridges which he built were of good design, substantial, and appropriate to their surroundings, and were constructed economically. There are in Boston many examples of the retractile draw-bridge, a peculiar type of movable bridge having certain advantages for low-grade crossings. This type was developed by Mr. Cheney from a crude and unscientific structure to one scientifically and economically designed, and remains as a distinctive feature of his work. He also perfected the wooden-leaf bascule bridge, and was among the first to build these bridges of iron. In connection with machinery for draw-

*Memoir prepared by the Secretary from information contained in an obituary notice for the Boston Society of Civil Engineers, by a committee composed of George F. Swain, E. D. Leavitt and Frederic H. Fay, Members, Am. Soc. C. E.

bridges and various mechanical plants his work gained for him a deserved reputation as a mechanical engineer. He had an extensive experience with tide-water foundations built under the peculiar conditions found about Boston, and was recognized as an authority in such matters.

He designed and superintended the erection of a great many bridges, among them being the Charlestown Bridge, constructed under the direction of the Transit Commission, and containing the widest draw-span in existence. The new Cambridge Bridge, between Boston and Cambridge, its graceful arches making it the most beautiful bridge of its kind in America, was probably his greatest work.

In addition to his work for the City of Boston, Mr. Cheney's abilities as a structural engineer brought him many commissions from outside parties. For a number of years he was Consulting Bridge Engineer for the Concord Railroad, the Boston, Concord and Montreal Railroad, and their successor, the Concord and Montreal Railroad, and designed most of the metal bridges on that system, and also the train-shed at Concord, New Hampshire. He was also Consulting Engineer for the Connecticut River Railroad and the Massachusetts Central Railroad. He was consulted by many counties, cities, and towns throughout New England in regard to highway bridges. His advice was also sought by many architects in reference to foundations and steel-frame buildings. Among such structures in Boston may be mentioned the Exchange Building, the Exchange Club, the Tremont Building, the Tremont Temple, the New England Conservatory of Music, and the Central Building. The silverware factory at Concord, New Hampshire, built by the William B. Durgin Company, is a meritorious example of mill construction designed by him.

Mr. Cheney always studied his problems with great care, but was never afraid to adopt a novel or untried form of construction or a new idea after he had convinced himself that it was right. His designs were all ingenious, economical, well thought out, and well adapted to the conditions of each case. He was soon recognized as an able engineer, and as taking rank with the best structural engineers in America.

In 1875 he was married to Ellen M. Neal, daughter of the Hon. Peter M. Neal, of Lynn. After returning to Massachusetts, he lived in Lynn for ten years; he then moved to Boston, and at the time of his death and for some time previous, had been living in the suburb of Brighton. He died suddenly, of heart trouble, on September 25th, 1906. Mrs. Cheney, a son, and a granddaughter survive him. His son, Herbert Neal Cheney, also a graduate of the Lawrence Scientific School, is following his father's profession, being at present with the Boston Consolidated Gas Company.

Mr. Cheney was a member of the Massachusetts Society and Boston Chapter of the Sons of the American Revolution, and of the Society of Colonial Wars in Massachusetts. He was also a prominent member of the Boston Society of Civil Engineers. He was elected a Member of the American Society of Civil Engineers on May 7th, 1884.

While of a very modest and retiring disposition, Mr. Cheney was, nevertheless, a man of great force and determination. All who knew him loved and respected him, on account of his high professional acquirements and his lovable nature and sturdy integrity. He was always thoughtful of the rights of others, and was a man who would never do a mean thing. His brother members in the Boston Society of Civil Engineers have written of him:

"His memory will long be kept green in the hearts of those who knew him, and his death leaves a vacancy in his professional as well as his personal circle which will not soon be filled. Men like him, so strong, so modest, so capable, so thoughtful, so kind, so helpful, are rare indeed."

JOSEPHUS CONN GUILD, M. Am. Soc. C. E.*

DIED FEBRUARY 25TH, 1907.

Josephus Conn Guild was born in Nashville, Tennessee, on January 4th, 1862. His father was Major George B. Guild.

His primary education was received from the Mims School, at Nashville, from which he entered Vanderbilt University, receiving from that institution M. A. certificates in the Scientific Schools in the summer of 1883. Immediately thereafter he was appointed Assistant State Geologist. Resigning from this office a year later, Mr. Guild accepted an engagement as Superintendent and Engineer of the Refugio Gold and Silver Mining Company, Durango, Mexico.

After nearly a year spent in the tropics, to which he always had an aversion, he returned to the States, securing a re-appointment as Assistant State Geologist.

From 1885 to 1887 he held the position of Inspector of Mines, State of Tennessee, forming, at the same period, the partnership of Dickerson and Guild, Civil and Mining Engineers. The firm operated also a chemical laboratory, and, locating at Chattanooga, Tennessee, soon established a satisfactory practice, numbering as clients several of the larger iron and coal companies of the region.

Mr. Guild resigned the State office in 1887, and at about the same time formed a new professional association—with Mr. Linn White—for the practice of civil and mining engineering, and engineering chemistry, under the firm name of Guild and White.

During this period of great activity and industrial development in and about Chattanooga, the young engineers executed most creditably many important commissions for the furnaces, mines, railways and development companies of the district.

During 1889, an important professional engagement was the design and construction of the Cameron Hill (Chattanooga) Passenger Incline Railway. Of similar character, but of far greater magnitude and importance, was the commission to build the passenger incline up Lookout Mountain, executed in 1894, and uniquely interesting in many engineering particulars, being in fact one of the longest and steepest inclines in the world. Its length was 5 000 ft., with an ascent of 1 700 ft., and with gradients of from 10 to 68 per cent. One of the most difficult problems, successfully solved, was the proper arrangement of the cableways along the sharp vertical curve where the railway location encountered the palisade or rocky escarpment, before reaching the Mt. Lookout plateau. The line is now in daily operation, and is well patronized.

* Memoir prepared by James Nisbet Hazlehurst, M. Am. Soc. C. E.

The necessity of undertaking the completion of an abandoned contract, upon which the firm had inadvisedly become bondsmen, turned the attention of Guild and White to the possibilities of municipal contract work, and thereafter, for a number of years, the firm name was a familiar one at contract lettings throughout the Southern States, and was synonymous with unvarying integrity and engineering skill of high order. Scores of water-works and systems of sewers were completed successfully by them, and the record was fully maintained and augmented, when, at the retirement of Mr. White, in 1896, the title of the firm was changed to Guild and Company, Incorporated, Mr. J. C. Guild being the first executive officer.

The corporation prospered greatly, growing in financial strength and engineering reputation, Mr. Guild's individual earnings being largely invested in the commercial, manufacturing and financial undertakings of his home city, Chattanooga. To his conservatism, energy, foresight, and shrewd business acumen, the firm prosperity and highly organized character of these institutions bear abundant witness, and stamp with the sterling mark this Captain of Industry.

The crowning achievement of his busy brain was his conception of the commercial possibilities which might accrue through the utilization of the great power of the Tennessee River.

Originally a project of the Army engineers for the improvement of the river's navigation, the potentialities of a hydro-electric installation in connection therewith were first recognized and made possible through Mr. Guild's knowledge of its engineering features, his strong personality, directed toward securing city, state, and national legislation, and his ability to obtain the necessary capital for this great undertaking.

At the time of his death Mr. Guild was largely interested in the financial outcome, and occupied the position of Chief Engineer to the Chattanooga and Tennessee River Power Company. This immense work is at the present time under construction.

Had he accomplished nothing beyond this undertaking he would have left to posterity a fitting monument to his memory. Originally intended by the Government as remedial work to facilitate slack-water navigation, it was proposed in 1900 to construct, some 30 miles below Chattanooga, a dam of heavy timber cribs filled with stone, the deck to slope downward each way from the crest. A lock of cut-stone masonry was also to be provided, the estimated cost being $1 000 000.

As a subsidiary proposition, Mr. Guild's conception was the practical utilization of the water-power at this dam and its conversion into electrical energy. In addition to the legislative and financial difficulties to be overcome, there were at least two most unusual engineering or physical problems to be solved. First, the distance or

height between the upper and lower pools could hardly be less at times than 50 ft., and was destined to be one of the highest single-lift locks in the world. The second difficulty and exceptional hydraulic condition to be met was caused by the most unusual variation in volume of flow and the resulting head upon the turbine wheels. To secure uniformity of speed and regular output under such conditions, three wheels are specified to each vertical shaft, the two lower turbines being fixed for the lower volumes, while the third wheel is intended to utilize great volumes at the lower head.

The lock, dam and power-house are of cyclopean concrete in continuous work. To convey some idea of the installation, it may be mentioned that the three cableways in operation span 2 300 ft., with vertical slack of 100 ft., constituting in themselves remarkable equipment features.

Such, in these poor words, are the activities and achievements of a lifetime.

Of splendid physique, clean and wholesome morals, a fascinating personality, and with a brilliant, trained and highly analytical mind, Josephus Conn Guild was called from Life's labors to enter into rest. He died on February 25th, 1907, in his home in Chattanooga, Tennessee, at the age of forty-five, in the prime of vigorous manhood, and at the height of his brilliant achievements.

Mr. Guild was elected a Member of the American Society of Civil Engineers on June 1st, 1898.

DUNKIN WIRGMAN HEMMING, M. Am. Soc. C. E.*

DIED MARCH 22D, 1906.

Dunkin Wirgman Hemming was born in Drummondville, Quebec, Canada, on November 10th, 1860, of a distinguished English Canadian family—his father, Edward J. Hemming, being a Queen's Counsel, and the first man to take the degree of Doctor of Civil Law at McGill University. He was named for an uncle, the Honorable Justice Dunkin, and another uncle was Senior Wrangler in mathematics at Cambridge University, England.

Mr. Hemming was educated at Bishop's College University, Canada. After graduation he studied law and was admitted to the Montreal Bar with high honors; he became associated with a prominent firm of criminal lawyers of Montreal, but gave up a promising career to avoid disagreement with his father, who disapproved of his connection with the criminal branch of legal practice.

He then turned his attention to Engineering, and, by his own unaided efforts, worked his way up to an enviable position as an engineer, and at the time of his death was without a superior, if he had an equal, in his special work as Contractor's Engineer.

In 1886 he entered the service of The Canadian Pacific Railway as Rodman, and was Transitman when he left that road, in 1889. His next engagement of length was as Assistant Engineer, and later as Resident Engineer, in charge of construction of the Baltimore Belt Railway, including the Baltimore and Ohio tunnel and extensive open work; he left this work in 1893, and, after a short engagement as Locating Engineer for The New York, New England and Northern Railway he came to New York City where practically all his subsequent work was done, the greater part of it as Contractor's Engineer with some of the strongest contracting firms and on the most important construction work in this vicinity, including such work as the construction of the Columbus Avenue and Lexington Avenue cable lines; the laying of many miles of gas mains and sewers; a part of the Park Avenue Improvements of the New York Central and Hudson River Railroad; the laying of pneumatic tubes connecting the New York Post-Office with Sub-Stations; putting in condensed air plants; laying water mains and changing sewers along the Rapid Transit Subway; piers for the Manhattan Bridge, 145th Street Bridge; Riverside Drive Extension, etc. In 1904 he became Engineer for one of the Contractors for the Rapid Transit Subway and the Jerome Park Reservoir, and was engaged in this work at the time of his last sickness.

He was married on June 4th, 1896, to Florence, eldest daughter

* Memoir prepared by Albert Carr, M. Am. Soc. C. E.

of Augustus W. Kriete, a wealthy and prominent citizen of Baltimore, and granddaughter of Judge Hugh Lennox Bond, of the old Maryland family of that name.

After about ten years of exceptionally happy married life he was attacked by an unexpected illness, which took him away at the time of his greatest usefulness, and after he had reached a position to reap the reward of his earlier labors.

A Canadian by birth, he became a naturalized citizen of the United States and a staunch American in ideals and beliefs, thoroughly loyal to the country which had afforded him wider opportunities than his native land.

Of a very companionable nature, and with the happy faculty of remembering people, he had a very large acquaintance in business and in the various organizations with which he was connected.

He was a member of the Franklin Institute, the Canadian Society of New York, and St. George's Society of New York.

He was elected an Associate Member of the American Society of Civil Engineers on September 7th, 1892, and a Member on May 31st, 1904.

CHARLES EZRA HEQUEMBOURG,* M. Am. Soc. C. E.

DIED OCTOBER 17TH, 1907.

Charles Ezra Hequembourg was the eldest son of the Reverend Charles Louis Hequembourg (Yale, 1836), a distinguished Presbyterian clergyman and author, whose grandfather, Charles Louis Hequembourg, came to America from Rouen, France, in 1778. His mother was Emelia Williams Hequembourg, a direct descendant of Stephen Williams. Under the wise guidance of his parents, he began at once to cultivate the qualities he had inherited, and these carried him through a remarkably successful career.

He was born on July 9th, 1845, in what was then the Village of Dunkirk, Chautauqua County, New York, which place he always made his home, though for forty years his business interests were mostly in other fields. His education was received in the public schools of Dunkirk and Dansville, New York, and Warren, Pennsylvania, in which places his father successively established his home and reared a family of seven children. Forced by circumstances, it was necessary for him to begin the battle of life while very young, and he earned his first wages working in the summer on the farm, and during the winter season making boats and skiffs to be used in the spring by the lumbermen of that vicinity. As he grew older, his interest in work of this nature quickened until he finally took to the trade of carpenter. At the age of 18, however, he laid aside his tools, and, with Company D, 68th Regiment, N. G. N. Y., went to the front and later joined the Army of the Cumberland.

When he received his discharge, at the end of the war, he began to acquire his engineering knowledge, and practiced in Tennessee and Kentucky, where he soon had charge of the field work, surveys, mapping lands, drilling wells, building pipe lines, etc., for the Tennessee and Cumberland Oil and Mining Company.

In 1870 Mr. Hequembourg returned to Dunkirk, and entered the contracting business. His first large contract was the erection of what is now School Building No. 2, in that city. The next year he erected the first brick schoolhouse in Titusville, Pennsylvania. During 1871 and 1872, while filling the office of Village Engineer, he constructed the Dunkirk Water-Works, a direct-pressure system, costing about $100 000. In 1873 and 1874, as engineer and contractor, he constructed the water-works systems of the Towns of Hyde Park and Lake, now part of the City of Chicago. These were also direct-pressure systems, and cost about $1 250 000. Later, at Bradford, Pennsylvania, he erected the St. James Hotel building, the second brick

* Memoir prepared by Thomas W. Symons, M. Am. Soc. C. E.

edifice that city possessed, and now its principal hotel. As a direct result of his location, he naturally became interested in oil developments, and was one of the early operators in the Pennsylvania field. He took special interest, however, in natural gas, of which he made an exhaustive study, and to the end of his life was an authority on all matters pertaining to the production and use of gas.

In 1878, as President and Engineer, with associates, Mr. Hequembourg organized the Bradford Gas, Light and Heating Company, and the Tarport and Kendal Gas Light and Heating Company. These were the first corporations to supply natural gas for light and fuel to a municipality. The original supply was obtained from natural pressure, but in 1880 this was superseded by a pumping station at Rixford, Pennsylvania, with a capacity of 6 000 000 cu. ft. per day. It was from the experience and knowledge thus acquired in the Pennsylvania fields that, in 1888, as President and Engineer of the Columbus Construction Company, he was able to undertake the task of connecting Chicago with the gas fields of Indiana, and in this undertaking he was most successful. In 1892, the Columbus Construction Company completed and turned over, to the owners of the Indiana Natural Gas and Oil Company, and the Chicago Economic Fuel Gas Company, what was then the largest and longest gas pipe-line system in the world. Fully equipped with modern pumping stations and appliances, this pipe-line is now in successful and profitable operation.

Mr. Hequembourg was strictly a self-made man. His parents, with a large family to rear, and with a limited income, were unable to give him the advantages he lived to give his sons and daughters, and from an early age he began to face the stern problems of life. He looked about him, saw his opportunities, and grasped them, and the secret of his success was that he always investigated thoroughly any matter which was of interest to him at the time. His mind was extraordinary, and drifted naturally into scientific channels, although he often remarked that his greatest ambition was to be a lawyer, and, in representing his associates or in his own work, he took actual delight in probing into the depths of the legal questions which were presented. He possessed a complete law and reference library, in addition to his collection of scientific works, and his friends often accused him of never being content unless he had a lawsuit on his hands.

After retiring from active practice, in 1892, he took up his home life at Dunkirk, and to gratify his longing for scientific research and knowledge, he erected near his residence an observatory consisting of a tower 65 ft. high, in the top gallery of which, mounted on a brick pedestal, was placed a 25-ft. telescope, with a 9-in. objective. This observatory is fitted with all modern appliances, instruments of precision, etc. On the other floors of the tower are arranged the library,

photographic room, and laboratory. It was here that Mr. Hequembourg loved to entertain his friends and enjoy his astronomical work until, unfortunately, in 1901, he slipped on an icy pavement and fractured his leg. After that he was not able to climb the tower stairs without considerable annoyance, and he was forced to give up his work in this field. Not content to remain idle, however, he became interested in automobile construction, and, a few years ago, took out a patent on an improved non-puncturable tire, but not until he had made many long and serious tests of his idea.

Mr. Hequembourg was a devoted and loving husband, and a kind and indulgent father, and was very happy in his home life, constantly striving to make his family comfortable and happy.

In March, 1904, he was selected by the people of Dunkirk as their Mayor. His election was not of a political nature, but was due rather to a movement for reform, and his administration was marked by an increase in local patriotism.

The following term he was on the ticket of both parties, and carried the city through another season of prosperity. His attention to municipal affairs was marked by the energy and earnestness with which he conducted his office. One of his fondest hopes was to live to see Dunkirk become a large city, and he believed that, on account of its natural advantages, it could be made so by proper co-operation. Dunkirk Harbor was the subject of special interest to him. At one time, at his own expense, he made surveys and soundings of this harbor, and brought about a public movement to get Congress to appropriate sufficient funds for the proper deepening of the channel and anchorage, and its protection by suitable breakwaters. This was accomplished, to a great extent, but Dunkirk needs another man, such as she has lost in Mr. Hequembourg, to come to the front and direct the gathering of the fruits of his labors in this field.

Mr. Hequembourg had been in excellent health all his life until last March, shortly after the death of a beloved son, Louis Max Hequembourg, a boy of 22 years, and a Junior in the College of Civil Engineering at Cornell University. The loss of this happy boy, whose future seemed so bright, and of whom he was so proud and hopeful, completely broke his heart, and from that day in March, when his boy was taken from him, he appeared to decline, and seven months afterward followed him to Fredonia, where he lies near him in the family plot in beautiful Forest Hill.

At St. Louis, in 1872, Mr. Hequembourg was married to Harriet E. Thurber, who survives him, with their son, Charles Guy Hequembourg, an engineer in New York City, and their four daughters, Mrs. J. L. Hurlbert, of Dunkirk, Mrs. F. K. Wing, of Buffalo, and the Misses Jessie and Hilda Hequembourg, who live at home.

Charles Hequembourg was a man of great activities. Always thoughtful, conservative, and unassuming, he possessed great determination, and once having mapped out a course of action, no obstacle seemed so great as to be unsurmountable. He planned and solved large business problems with the skill of a master, in the fulfillment of which he crossed swords with many of the great captains of industry, and in the action was seldom, if ever, worsted. He commanded the respect of his associates, and all men with whom he came in contact, no matter on which side they stood. A large man, physically, he possessed a heart large in proportion, and was always sympathetic, kind, and courteous to all. He was surely one of God's noblemen. He possessed, in a marked degree, the qualities of self-reliance, courage and inflexibility of purpose. If these characteristics are applied to the branches of activity wherein his energy found an outlet, it is easy to understand why he was a successful engineer, and an instrument in the development of large interests, and why he has left behind him so many who will mourn his loss.

Mr. Hequembourg was a Thirty-third-degree Mason of the Ancient Order of the Accepted Scottish Rite. He was elected a Member of the American Society of Civil Engineers on June 5th, 1901, and, although he took no active part in Society matters, he was a close observer of all that transpired, and took the greatest pride in his membership.

JACOB HAYS LINVILLE, M. Am. Soc. C. E.*

DIED AUGUST 4TH, 1906.

Jacob Hays Linville, of Pennsylvania Quaker ancestry, was born on September 23d, 1825, on the farm of his father, Arthur Linville, near the little village of Pequea, Lancaster County, Pennsylvania. His father, besides farming, was also engaged as a tanner and currier, at Gap, Pennsylvania. His mother, also a Pennsylvanian, was Elizabeth Haines.

His primary education was obtained in the local public schools, and was followed by a course in the Academy of Dr. Duffield at Belleview, Lancaster County, Pennsylvania, where he was prepared for his college entrance examination. He was matriculated at Union College, Schenectady, New York, in September, 1845, at the age of twenty, and was graduated, with the degree of B. A., in July, 1848.

Nothing has been learned to indicate that he had any predilection for engineering as his future profession; in fact, shortly after his graduation, he engaged in teaching, in his home public schools, for a short term only, and then he seems to have decided to adopt the Law as his profession, and, with that in view, in 1851 or 1852, he entered, as a student, the law office of William M. Meredith, Attorney at Law, in Philadelphia. Evidently, however, he discovered very soon that the Law was not his vocation, for he terminated his studies in it in less than a year.

Then, apparently, he determined to adopt Engineering, and his first employment was as a topographical assistant in the corps of engineers making the surveys for the Lancaster, Lebanon and Pine Grove Railroad, in his home county. This was followed by an engagement as Assistant Engineer in the corps of the late William Hasell Wilson, Hon. M. Am. Soc. C. E., then engaged upon the location and construction of the Philadelphia, Media and Westchester Railroad. In 1857 he was appointed Assistant Resident Engineer, under Mr. Wilson as Resident Engineer, on the Middle Division (Harrisburg to Altoona) of the Pennsylvania Railroad.

On May 21st, 1857, Mr. Linville married Miss Celeste Virginia Rush, daughter of a Philadelphia merchant. Their union was blessed with only one child, a son, named Samuel Rush Linville, who in later life adopted as his profession that of "Electrician;" he died in Philadelphia on May 12th, 1899, in his forty-first year.

In 1863 Mr. Linville was appointed Engineer of Bridges and Buildings for the Pennsylvania Railroad, with residence and office

* Memoir prepared by Walter Katte, M. Am. Soc. C. E.

in Altoona, Pennsylvania, the general headquarters and offices of that company's Operating and Construction Departments. This appointment seems to mark a singularly interesting and important turning point in Mr. Linville's professional career, and is apparently the initial one from which started his subsequent brilliant achievements in the specialties of design and construction of railroad bridges and buildings, remarkable for the reason that, as far as can be learned, he started with very little, if any, technical training or practice in those specialties. That he fully realized this, is disclosed by the following extract from a letter written by him to a close personal friend many years later:

"I went to Altoona with a young wife, with everything new to me. The bridges on the line were nearly all a wreck; I knew nothing of bridges, and had at my disposal nothing but Haupt's old book, all wrong. Orders came in to build new iron bridges, and I had to hustle, with many sleepless nights, and days spent over old patterns and new plans and calculations involving many changes and improvements in the plans of the original cast-iron chord, arch, and posts of the old Pratt type."

In 1863-64, the Pennsylvania Railroad Company having determined to prosecute the construction of the old Pittsburg and Steubenville Railroad, the question of bridging the Monongahela River at Pittsburg, and the Ohio River at Steubenville, at once became of very serious moment, as it was most energetically and violently objected to by the united coal and boating interests of those rivers. It was alleged, and not without a good deal of plausibility, that any piers placed in their boating channels would inevitably and utterly destroy their business, by rendering it impossible, or so dangerous as practically to make it impossible, to continue the old-fashioned "broadhorn" system of floating their coal to market, on which, to a great extent, it depended. This controversy was stubbornly maintained on both sides, and was fought with most virulent animosity, through no end of "Courts" and "Legislatures," until, finally, a compromise was arrived at when the Railroad Company agreed to construct, over the navigable channels of the rivers, spans of not less than 300 ft. in clear between the piers, on the water line.

The river interests believed that the railroad company would find it impracticable to design and erect a span of that length to sustain safely the then prevailing weight of railroad trains, as, at that time, it was pretty generally believed, even by many technical experts, that 250 ft. was about the safe limit of spans of bridging to sustain the railroad "live loads" of that day. However, the President of the Pennsylvania Railroad Company at that time—Mr. John Edgar Thomson—was far too astute, and too good an engineer himself, to have ever agreed to such terms, had he not

previously assured himself of perfect ability to comply with them. He had been for some years well acquainted with Mr. Linville, and entertained a high opinion of his abilities as a bridge designer, and had accepted his assurance that it was practicable to design metallic spans, from 300 to 350 ft. in length, with entire safety.

Mr. Linville made only one reservation, namely, that the railroad company should cause to be built for him a testing machine sufficiently powerful to "test to failure" full-sized bridge members. Mr. Linville had satisfied himself—as indeed had many other engineers of that day—that the old formulas and experiments of Hodgkinson, Fairbairn, Kircaldy, *et al.*, could not be relied on to give absolutely safe standards of the ultimate strength and moduli of elasticity of full-sized members, as they were derived from the results of laboratory tests made upon samples of very small sectional area and length, and were entirely lacking in proof that the ratios would hold good when applied to the areas and lengths of full-sized members. Mr. Thomson, fully appreciating the value of this "proviso," at once gave Mr. Linville *carte blanche* to have Messrs. William Sellers and Company, of Philadelphia, build such a testing machine as would fulfill all his requirements. Such a machine was built by that firm, and was set up in the railroad company's shops at Altoona, and, in all probability, was the first testing machine ever built of sufficient power to demonstrate the actual limits of ultimate strength and elasticity of full-sized metallic members of framed structures.

With this machine Mr. Linville conducted a series of tests from which he derived reliable data from which to assign the necessary safe sectional areas for the members of "long-span" structures. This machine, having thus filled the requirements for which it was specially built, was subsequently bought by the Keystone Bridge Company, and re-erected in that company's Pittsburg shops. It is probably still in the service of the successors of that company, the American Bridge Company, now merged into the United States Steel Company. Thus did this Steubenville span of 320 ft. become indeed the "pioneer" of long-span bridging in the United States, which fact is attested to in the following quotation from the admirable paper* on "American Railroad Bridges," by Theodore Cooper, M. Am. Soc. C. E.:

"The era of long span truss bridges in America may be considered as dating from the building of the first bridge over the Ohio River at Steubenville, between 1863-64, by Mr. J. H. Linville. The channel span was 320 feet long and 28 feet deep. The top chord and posts were made of cast-iron. It was proportioned for a rolling load of 3 000 pounds per foot of track, a notable increase in the load heretofore in use."

* *Transactions*, Am. Soc. C. E., Vol. XXI, p. 1.

In the latter part of 1864, Mr. Andrew Carnegie procured the incorporation, under the laws of Pennsylvania, of the Keystone Bridge Company, to take over the shops and business of the bridge building firm of Piper and Shiffler, of Pittsburg, at that time engaged principally in manufacturing and erecting the reconstructed bridges of the Pennsylvania Railroad upon the plans and specifications of Mr. Linville. This new company was organized with Mr. Linville as President and Consulting Engineer; John L. Piper, General Manager; Aaron G. Shiffler, Treasurer and General Superintendent, and Walter Katte, Secretary and Engineer of Shops and Erection.

Ample new capital being subscribed, the business scope and shop manufacturing facilities were at once largely augmented, resulting in this company speedily taking prominent rank among the leading bridge building organizations of the country. To this company the Pennsylvania Railroad Company awarded the contract for the manufacture and erection of the superstructures of the Steubenville and Monongahela Bridges, upon the plans and specifications previously prepared by Mr. Linville and accepted by the railroad company. The successful execution of this order, and the experience gained therefrom, so qualified this company and established its reputation as builders of long-span metallic bridges, that it at once entered the field as a very active competitor in the many bridging projects of the great western rivers, then being promulgated, all requiring exceptionally long spans. To this company were subsequently awarded the contracts for the manufacture and erection of the superstructures of the following notable long-span bridges, all of which were designed by Mr. Linville:

Monongahela River, Pennsylvania Railroad, at Pittsburg, Pa.;

Ohio River, Baltimore and Ohio Railroad, at Bellaire, Ohio;

Ohio River, Baltimore and Ohio Railroad, at Parkersburg, W. Va.;

Ohio River, Cincinnati and Newport Railroad, at Cincinnati, Ohio;

Ohio River, Cincinnati Southern Railroad, at Cincinnati, Ohio;

Missouri River, Draw Span, Hannibal and St. Joseph Railroad, at Kansas City, Mo.;

Mississippi River, Illinois Central Railroad, at Dubuque, Iowa;

Mississippi River, Keokuk and Hamilton Bridge Company, at Keokuk, Iowa;

Mississippi River, Draw Span, Chicago and Alton Railroad, at Louisiana, Mo.

The shop detailed working drawings of the great steel arch bridge over the Mississippi River at St. Louis, designed by the late James B. Eads, M. Am. Soc. C. E., and manufactured and erected by the Keystone Bridge Company, were also reviewed and revised

personally by Mr. Linville in order to insure the successful erection of that great and novel structure, upon the cantilever suspension system, for which the Keystone Bridge Company was under contract responsibility.

During his service as Engineer of Bridges and Buildings of the Pennsylvania Railroad Company, Mr. Linville designed and supervised the erection of the new wrought-iron bridges over the Schuylkill River, at the Arsenal, Philadelphia, on the Delaware River extension of the Pennsylvania Railroad. This was the first bridge built with wrought-iron posts, and "upset" head-link tension members, and, by its use of hollow wrought-iron columns, started the long legal contest between the Phœnix Iron Company and the Keystone Bridge Company *et al.*, for alleged infringement of its patent rights. This was finally decided by Justice McKenna, in the United States District Court of Pittsburg, in favor of the defendant parties.

At about this time Mr. Linville resigned from the service of the Pennsylvania Railroad Company, and removed to Philadelphia where he opened an office for private practice as Consulting and Designing Civil Engineer. He was immediately commissioned by the Pennsylvania Railroad Company to prepare plans and specifications for the reconstruction in metal of many of its most important bridge structures, among which may be specially noted the following: "Dauphin Bridge," Susquehanna River, on the Northern Central Railroad; and the "Maryland's Creek" and "Ridley's Creek" Bridges, on the Media and Westchester Railroad; and as Engineer of Bridges, for the "Connecting Railway" of Philadelphia, extending from the main line (Pennsylvania Railroad) at Mantua, to the New York Division at Frankfort, he designed the double-track, under-grade bridge over the Schuylkill River, near Girard Avenue; and about the same time (1875) he designed for the City of Philadelphia the new Callowhill Street Bridge, or "Fairmount Bridge," over the Schuylkill River, replacing the old wire Suspension Bridge at that place. It is a double-deck, rectangular-truss structure, 348 ft. in length, between end posts, and provides for double-track electric street railway tracks, carriage roadways, and sidewalks, on both decks. He was also commissioned by the Lehigh and Delaware Railroad Company to prepare plans and specifications for the bridging of its line, including notable structures over the Delaware and Lehigh Rivers at Easton, Pennsylvania; and also by the Central Railroad Company of New Jersey for its long-span pivot draw-spans at Amboy and Newark, New Jersey.

In addition to his high attainments as a bridge designer, Mr. Linville developed marked skill as an architect, notably in the

specialty of industrial buildings. Having a finely cultivated artistic taste, his designs for such buildings, while skillfully adapted to the technical requirements, were always architecturally attractive in their treatment. He was commissioned by the Pennsylvania Steel Company as its Chief Engineer during its constructive period, 1863-65, and designed and supervised the erection of all its shops and buildings at Steelton, Pennsylvania. For the Pennsylvania and the Philadelphia and Erie Railroad Companies he designed the shops at West Philadelphia, Harrisburg, Sunbury, Renovo, and Kane; also those at Uhrichsville for the Pittsburg, Cincinnati and St. Louis Railroad Company, and the new shops for William Sellers and Company, in Philadelphia.

In the early days of elevated railroad projecting Mr. Linville was consulted by Dr. Gilbert (one of the original promotors of elevated railway projects) to revise and perfect his plans, which resulted in his designing the structure which was subsequently manufactured and erected for the Metropolitan Elevated Railroad Company (afterward the Manhattan Elevated Railway Company) on its Sixth Avenue line, between Canal and Fifty-ninth Streets, New York City.

In 1876 Mr. Linville made a notable exhibit of engineering drawings and models at the Philadelphia Centennial Exposition, for which he was highly commended, and received a bronze medal.

About the year 1880 Mr. Linville and A. P. Boller, M. Am. Soc. C. E., were selected by the Pennsylvania Railroad Company to act as arbitrators in the settlement of a controversy between that company and the Long Branch Railroad Company. Subsequent to this Mr. Linville was induced to assume the Presidency of a railroad company which had failed in its efforts to complete a railroad in the southwesterly peninsula of Maryland. He took hold of this moribund enterprise, and by his untiring energy was successful in clearing off all its old debts and judgments; he called in and cancelled all its old stock and bond liabilities, issued new stock, and effected a contract with a Boston syndicate to complete the construction and equipment of the road. His efforts in this field, however, were finally frustrated by the failure of the financial promotors of the enterprise to support it.

In the latter days of his professional life, Mr. Linville was induced, by parties owning patents on an electro-magnetic printing telegraph device, to undertake to perfect their machine, as hitherto they had failed to make it work satisfactorily. He became most enthusiastically interested in it, and worked over it with such energy, that, in the end, his health was seriously affected, but finally he succeeded in perfecting the apparatus. It is claimed that his was the first machine, of this type of electrical telegraphic devices, that

worked perfectly. In the end, however, it was a great loss and disappointment to him, owing to the failure of the promoting company to finance the enterprise adequately.

For many years prior to his death, Mr. Linville was a most acute sufferer from chronic catarrhal bronchitis and neuralgia of the head, which affected his eyes. Toward the end, his sufferings from these causes became so intense that he was compelled to give up his professional practice. Personally, his disposition was singularly loving and sympathetic, and all with whom he came in contact were greatly attached to him.

Mr. Linville was elected a Member of the American Society of Civil Engineers on March 3d, 1875.

GEORGE BENSON NICHOLSON, M. Am. Soc. C. E.*

DIED DECEMBER 2D, 1906.

George Benson Nicholson was born in Cheltenham, England, on September 24th, 1840. His father, John Nicholson, was an Irishman of English descent, and his mother, Matilda Jane Williams, was Irish by birth, but Welsh and English by descent.

The family came to America in 1842, and after a short time located in Cincinnati, Ohio.

The father died in 1852, and the care and support of the family thereafter devolved upon George, the only son, and his sisters. Under these circumstances, there was not much time or opportunity for the boy to obtain a school education, and this was limited to the common schools, and one year in the high school of Cincinnati. But, all his life, he was a close student, and an extensive and discriminating reader, and thus largely made up the deficiencies in his early education. As illustrating this, it may be mentioned that he was familiar with the classics of his own tongue, mastered the French language, and was considered an authority upon heraldry —a specialty of which he was particularly fond.

He began the serious business of life at the age of sixteen, when he entered the office of the late A. W. Gilbert, at that time one of the leading engineers of Cincinnati, and thus commenced the special work and studies which were to lead to his professional career.

When the Civil War came on, young Nicholson was one of the earliest to volunteer as a soldier in the Union cause. He enlisted on April 22d, 1861, in Company K, Sixth Ohio Volunteer Infantry (widely known as the Guthrie Greys), and on June 18th he was enrolled in that regiment for three years, and served until the expiration of his term.

Of his career as a soldier, the following endorsement on his discharge papers affords a brief but eloquent record:

"HEADQUARTERS CO. K.,
"6TH REG., O. V. I.
"June 23rd, 1864.

"First Sergeant G. B. Nicholson deserves more than an ordinary notice of the meritorious service he performed during his term of enlistment. Although but a mere youth when he enlisted, he bore the fatigues of the arduous campaigns participated in by his regiment in West Virginia, Kentucky, Tennessee, Mississippi, Alabama and Georgia, with a degree of fortitude seldom found in one of his age and physique. He was ever a brave, intelligent, uncomplaining

* Memoir prepared by Samuel Whinery, M. Am. Soc. C. E.

soldier, and behaved with more than ordinary gallantry in the engagements of Carrick's Ford (July, '61), Shiloh, Stone River, Chickamauga and Missionary Ridge. At the battle of Stone River he received a desperate wound, from which it was feared he would not recover; but his wonderful courage and determination carried him through. I deem this tribute the just due of one whom it was an honor to command, and whom I consider the beau-ideal of an American volunteer soldier.

"(Signed) J. M. DONOVAN,
"Capt. Co. K, 6th O. V. I."

His patriotic duty having thus been discharged, he again turned his attention to the profession he had chosen for his life work.

For a year he was engaged as Assistant Engineer in the military service of the Government, Department of Kentucky; then, for a short time, he was Mechanical Draftsman in the office of a patent attorney in Cincinnati. This was followed by 6 months' service as Assistant Engineer on street railroad construction in the same city. In March, 1866, he went to what was then the wilds of Montana and Idaho to locate and construct the Lewiston and Virginia City wagon roads, one of which he located through the Bitter Root Range of the Rocky Mountains; but, owing to inadequacy of funds, little construction work was done.

Returning to Cincinnati in March, 1867, he entered the service of the city as Engineer in Charge of Sewers, but in October of the same year he gave up this work to re-enter the Government service as United States Assistant Engineer on the improvement of the Mississippi and the Missouri Rivers.

Early in 1869 he became Assistant Engineer of the Fifth Light-house District, and spent two years in charge of the construction and repair of light-houses, wharves, etc., along the coasts of Maryland, Virginia, and North Carolina.

From February, 1871, to February, 1872, he was Shop Inspector, for the Government, of the ironwork for the Rock Island Bridge. He then returned to the Light-house service, in the Eighth Light-house District (coast of the Gulf of Mexico), where he had charge of the design, construction, and repair of light-houses, keepers' dwellings, beacons, breakwaters, and other work incidental to the position, and the making of several hydrographic surveys.

On July 4th, 1873, he entered upon the work that was chiefly to occupy the remainder of his life—the location, construction, and engineering maintenance of the Cincinnati Southern Railway and its associated roads. At first he was engaged upon the surveys for the location of the Ohio River Bridge and its approaches at Cincinnati.

Then, in June, 1874, he became Division Engineer in direct charge of the construction of the first 40 miles south of Cincinnati,

involving a large amount of difficult and expensive work. As the construction work on other divisions to the southward approached completion, Mr. Nicholson's jurisdiction was extended over them, and, for several years, after November, 1876, he had charge of the engineering work of the northern 160 miles of the road, under the late L. F. G. Bouscaren, M. Am. Soc. C. E., Chief Engineer.

The building of the Cincinnati Southern Railway was one of the notable public works of that period, and marked many substantial advances in railroad engineering. The Ohio River Bridge contained the longest truss-span attempted up to that time, and the Kentucky River Bridge was not only the highest railroad bridge in the world, but was the first cantilever bridge in America. The designs and specifications for the bridges for this road mark an epoch in the progress of bridge building. The road was a difficult and expensive project. The stretch from the Cumberland River to the Emory River, crossing the Cumberland Mountain, was probably the heaviest and most expensive one hundred consecutive miles of railroad ever built in America. There were twenty-one tunnels within the limits named. The construction work was begun under the late Thomas D. Lovett, M. Am. Soc. C. E., as Chief Engineer, with the late L. F. G. Bouscaren as first Assistant. When Mr. Lovett retired, Mr. Bouscaren succeeded him, concentrating upon the work his unflagging energy and his brilliant mind. The work afforded a valuable school of experience for the younger men engaged upon it, several of whom, in addition to Mr. Nicholson, have since become well known in the profession.

In 1880 the lessees of the Cincinnati Southern Railway and the Alabama Great Southern Railroad, decided to build the New Orleans and North Eastern Railroad from Meridian, Mississippi, to New Orleans, Louisiana, and Mr. Nicholson, under Mr. Bouscaren, Chief Engineer, was placed in charge of the location and construction of the southern half, from New Orleans northward for about a hundred miles, and to this work he devoted the next four years. The longest railroad trestle bridge then in existence, 26 miles, carrying the road over Lake Pontchartrain and its marshes, was built under Mr. Nicholson's immediate charge. This short line of road involved other difficult and troublesome work quite out of proportion to the magnitude of the enterprise.

In 1885 Mr. Nicholson succeeded Mr. Bouscaren as Chief Engineer of the Cincinnati Southern and Associated roads, known then as the Queen and Crescent System. While changes in organization and in the constituent roads, which make up the present Queen and Crescent System, took place in the meantime, Mr. Nicholson continued to be its Chief Engineer until his death, retaining the unbounded confidence and friendship of every manage-

ment. It is currently believed that during this time he was more than once offered promotion to higher executive departments, but no official confirmation of the fact is at hand, and Mr. Nicholson's extreme modesty prevented him from discussing such matters even with his intimate friends. It is known that outside positions, involving high honor and attractive compensation, were offered to him during this period, but he remained loyal to the road to which his services had become almost indispensable. Having been connected with the system since its inception, he knew every engineering and historical detail of its construction and operation, and could therefore serve the management far more efficiently than any new man who might be called to take his place.

His services, however, were not confined to the roads of which he had official charge. He was one of the trusted lieutenants of the late Samuel Spencer, President of the Southern Railway, of which the Cincinnati, New Orleans and Texas Pacific Railway is a part, and he was frequently called upon confidentially to examine and report upon important matters outside of his official jurisdiction.

To these, as well as to his own legitimate work, he applied and spent himself without reserve, and with little regard for his physical and mental welfare. Like others of his intense and conscientious temperament, he devoted his personal attention to many details which he might more wisely have referred to his assistants, who were able and loyal. The late night, as well as the day, usually found him at his desk. The result was the too frequent one. The constitution that had carried him safely through the hardships and the physical stress of an unusually strenuous service in the Army, finally gave way under the later and greater mental and physical strain, and he found himself face to face with a nervous collapse. Early in 1906, against his will, he was directed by his superior officers to take a vacation. Returning from this, not much improved in health, his leave of absence was extended indefinitely, and he was persuaded to go abroad on full pay, with the hope that his health might be restored. This hope was not realized. His health continued to decline, and, returning late in November, 1906, he survived but a short time, dying on December 2d, 1906, in the home in Covington, Kentucky, he had so long occupied.

Of Mr. Nicholson's personal character and professional ability and attainments, it may be said that against his honor and integrity there was never a breath of suspicion. Considering the fact that his ordinary educational advantages were very limited, and that of technical school education and training he had none, his knowledge of the technical side of engineering was quite remarkable; but he was most noted for his sound practical sense and good judgment —the result of an able and alert mind backed by a wide experience.

To his extreme modesty, amounting almost to shyness, in professional as well as in social affairs, must be attributed the fact that he appeared but seldom in the public eye.

No memoir of Mr. Nicholson, however brief, would be complete without some reference to his connection with "The Literary Club," of Cincinnati. This organization, founded in 1849, has been a notable center of literary culture in that city. Its membership list at any period, since its organization, has embraced the leading public, professional and literary men of the city and the surrounding region. In this list, past or present, may be found the names of one President of the United States, nine Generals of prominence in the United States Army, two United States Senators, three Governors of Ohio, seven Members of Congress, one Chief Justice of the United States, one distinguished actor, no less than three well-known poets, one Minister to Russia, and the present very able and honored Secretary of War.

Its meetings, held every Saturday night, when a paper is read, followed by a modest collation and an hour of social intercourse, may, without exaggeration, be characterized by the phrase, "A feast of reason and a flow of soul."

Mr. Nicholson became a member of this organization in 1885, and thereafter, until his death, was one of its most devoted and useful members, serving as its President for two years, and for many years in other official capacities. In the cultured bonhomie of this club he found a congenial atmosphere in which the fine qualities of his mind and heart expanded into full flower. Next to his family and his work, this club and its interests occupied a front place in his mind and activities during the later part of his life. It is stated that "He contributed some twenty papers to the club, every one being a paper of merit." It was principally through his efforts that all the published works, numbering between 300 and 400, of those whose names appear on the membership list of the club since its organization, were collected into the club library.

The verdict of such a body of men as compose the membership of this club, upon the personal character and worth of Mr. Nicholson may be accepted as conclusive. The minute adopted by the club in his memory is, therefore, here in part quoted:

"His chief characteristics were integrity, a high sense of honor, modesty and unselfishness; the latter manifesting itself in a kindly interest, not only in people, but in the lower animals. If he encountered a tramp dog lying shivering on a cellar door, he would go out of his way to a chop house, buy food and carry it to the suffering creature.

"Everything he undertook to do he did well, he neglected nothing; and the only reward he valued was the consciousness of duty fulfilled and honor cherished. He had no itching greed for wealth

nor social prominence, but enjoyed the amenities of life with his groups of friends with hearty good will. Owing to his devotion to his mother and sisters, and the children of one sister who was left in middle life with a dependent family, he never married.

"No one of us ever heard him refer to his military service, nor to any of the great achievements of his life. His modesty forbade; but he preserved all the records of his career as the safeguards of his honor. He felt, with Norfolk:

> "'Mine honor is my life;
> Both grow in one;
> Take honor from me,
> And my life is done.'

"The exquisite charm of his friendship can be known only to those who enjoyed it; to them it was a benediction, and his death an irreparable loss."

Others who had the honor and pleasure to be numbered among his more intimate friends will fully endorse this estimate of his character.

Mr. Nicholson was elected a Member of the American Society of Civil Engineers on May 1st, 1878, took a keen interest in its affairs, and occasionally contributed to the discussion of papers read before it.

MERRITT HARRISON ROGERS, M. Am. Soc. C. E.*

DIED MAY 3D, 1907.

Merritt Harrison Rogers was born on May 19th, 1851, at Center Moreland, Pennsylvania, of one of the oldest New England families, his ancestors having been among the first settlers in Salem, Massachusetts.

He was educated at New Columbus Academy and Wyoming Seminary.

Mr. Rogers took part in the location and construction of several railroads in Pennsylvania, and in 1878 entered the service of the Kansas Pacific Railroad. In 1880 he became Transitman on the Denver and Rio Grande Railroad, and was appointed Assistant Engineer of that road in 1881 and Division Engineer in 1883, which position he held for six years. At that time the office of Chief Engineer was abolished, and Mr. Rogers was made Resident Engineer, with the duties and powers of Chief Engineer. Many extensions and improvements were made during this time.

In 1890, the office of Chief Engineer was revived, and Mr. Rogers was appointed to that position. In 1892 the superintendency of bridges and buildings was added to his duties.

In 1902 he retired and became a Consulting Engineer. At the end of two years he went to California for his health, and remained there a year. He returned to Colorado about a year ago and was appointed Chief Engineer of the Denver and Transcontinental Railroad. While going into the field to start a locating party for the season, he was stricken with apoplexy and died.

Mr. Rogers possessed the sterling qualities of his New England ancestors. He was of a high personal character, devoted to his friends and to his profession. He was an engineer of ability, and did much to develop the railroads of the West.

About twenty-two years ago he married Miss Mattie Dodge, of Buffalo, New York, who survives him. They had two children, but both died in infancy.

Mr. Rogers was elected a Member of the American Society of Civil Engineers on January 2d, 1890.

* Memoir prepared by J. W. Deen, M. Am. Soc. C. E.

ALBERT JOHNSTONE CAMPBELL, Assoc. M. Am. Soc. C. E.*

DIED MARCH 23D, 1907.

Albert Johnstone Campbell was born at Stoke, England, on December 7th, 1854, and was the last survivor of the ancient family of Glen Saddel. As lineal heir of the Earls of Crawford and Lindsay, he was claimant to the dormant Earldoms of Annandale and Hartfield.

After being educated at Rugby, he entered the Royal School of Mines, passing successively through the Chemical, Metallurgical, Geological and Physical Sections, and being awarded the Associateship of Mines in 1876. The next four years were spent in the study of practical mining and engineering in Cornwall and Spain, and in 1880 he was appointed General Manager of the Trojes and Anjangeo Silver Mines and Smelting Company, in the State of Michoacan, Mexico, which position he held for three years, carrying out the successful development of the mines, and introducing many improvements in the system of smelting and general organization of the works.

In 1883, Mr. Campbell joined Mr. H. V. Rudston Read in a partnership, as Consulting and Constructive Engineers, with a branch establishment in Mexico, and, while in that country, carried out many important engineering works, for the Mexican Government and by private contracts, amounting to many millions of dollars. The principal work, and the one with which Mr. Campbell's name will always be associated, was the construction of the Tequixquiac Tunnel for the drainage of the Valley of Mexico, the length of the tunnel being 6¼ miles—at that time the longest tunnel in existence.

The splendid organization and ability displayed by Mr. Campbell in carrying through this work in the face of the greatest difficulties was recognized officially by the President and Government of Mexico, and, at a later date, the Committee of the Chicago Exhibition presented Mr. Campbell's firm with a gold medal as some recognition of the magnitude and importance of the work, and of the skillful engineering displayed in its performance. Also, in conjunction with his partner, he built the Mexican Southern Railway, which was the first railway in Mexico to cross the Sierra Madre Mountains, and, up to that date (1892), was the only railway in Mexico which had been completed from the start with permanent work in every detail. The time allowed for the construction of this road was ten years, but it was completed in three years, and a bonus of $1 800 000 was earned on that account. The railroad was fully ballasted, with all masonry, culverts, bridges, etc., in position. One-third of this line is probably one of the heaviest pieces of railroad work on the North American Continent,

* Memoir prepared by F. W. Abbot, M. Am. Soc. C. E.

60 miles of it being constructed in a cañon with 4% grades. In its construction it was necessary to build 150 miles of side-roads to carry in supplies to the 8 000 men employed on the work.

The characteristic zeal and energy with which Mr. Campbell threw himself into these and other works in which he was engaged while in Mexico led to the serious undermining of his health; in fact, the illness which necessitated the operation from which he subsequently died was directly traceable to the fevers contracted twenty years before in the Mexican swamps.

Mr. Campbell was elected an Associate Member of the American Society of Civil Engineers on May 2d, 1894. He was also a Member of the Institution of Mechanical Engineers, a Member of the Institution of Civil Engineers, and an Associate of the Royal School of Mines.

JOSEPH WILLIAM ZIPPERLEIN, Assoc. Am. Soc. C. E.*

DIED JULY 22D, 1907.

Joseph William Zipperlein, born in Philadelphia, Pennsylvania, on February 13th, 1870, was the youngest son of Charles and Louisa Zipperlein. He attended the public schools of Philadelphia until 17 years old, when he left school to accept a position in the publishing house of Lea Brothers. After about one year's service he resigned, to begin, in November, 1888, his career in the cement business, entering the employ of the Commercial Wood and Cement Company as stenographer, from which position he gradually advanced to that of Secretary in 1891. At this time he withdrew from the company in order to become identified with The William G. Hartranft Cement Company, as Vice-President, a position which he held continuously until his death.

Mr. Zipperlein was closely identified with the development of the American Portland cement industry, being a pioneer salesman, not only in the United States, but also in Canada; and, in the latter country, he has been financially interested in the cement industry since 1898.

Mr. Zipperlein was beloved by all who knew him. He was of modest disposition, amiable, tactful, a staunch friend, and a thoroughly honorable business man. Quiet and even tempered, honest in all his dealings, resourceful in his work, he commanded, not only the entire confidence and respect of his business associates, but of his employees as well.

In the summer of 1906 sickness forced him to go abroad, and, although the trip proved beneficial, he never fully regained his health, but suffered almost continuously until his death, on July 22d, 1907.

He was elected an Associate of the American Society of Civil Engineers on June 4th, 1901. A large number of friends and acquaintances mourn his untimely death.

* Memoir prepared by Richard L. Humphrey, M. Am. Soc. C. E.

TRANSACTIONS

OF THE

American Society of Civil Engineers.

INDEX.

VOLUME LIX.

DECEMBER, 1907.

SUBJECT INDEX, PAGE 568.

AUTHOR INDEX, PAGE 573.

Titles of papers are in quotation marks when given with the author's name.

VOLUME LIX.

SUBJECT INDEX.

BRIDGES.

Effect of San Francisco earthquake on — . 258.

Effect of San Francisco earthquake on highway structures. 263, 270, 311, 323.

"The Reinforced Concrete Bridge across the Hudson River at Sandy Hill, New York." William H. Burr. 195. Discussion: Henry H. Quimby, C. L. Slocum, D. W. Krellwitz, and E. W. Stern, 203.

BUILDINGS.

"The Effects of the San Francisco Earthquake of April 18th, 1906, on Engineering Constructions": Reports of a General Committee and of Six Special Committees of the San Francisco Association of Members of the American Society of Civil Engineers. (With Discussion.) 208.

CONCRETE.

"The Laws of Proportioning — ." William B. Fuller and Sanford E. Thompson. 67. Discussion: James L. Davis, H. de B. Parsons, Allen Hazen, Samuel Tobias Wagner, R. Feret, W. F. Dennis, and Richard H. Gaines, 144.

CONFLAGRATIONS.

"The Effects of the San Francisco Earthquake of April 18th, 1906, on Engineering Constructions": Reports of a General Committee and of Six Special Committees of the San Francisco Association of Members of the American Society of Civil Engineers. (With Discussion.) 208.

DAMS.

Effect of San Francisco earthquake on earth and concrete — . 245.

DISEASE.

"Water Supply." An Informal Discussion. George W. Fuller and others. 367.

EARTHQUAKES.

"The Effects of the San Francisco Earthquake of April 18th, 1906, on Engineering Constructions": Reports of a General Committee and of Six Special Committees of the San Francisco Association of Members of the American Society of Civil Engineers. 208. Discussion:

Edwin Duryea, Jr., Franklin Riffle, H. H. Wadsworth, E. G. Perrot, Luther Wagoner, Eugene W. Stern, J. K. Freitag, W. W. Harts, E. A. Rix, W. F. Whitaker, A. M. Bienenfeld, Arthur L. Adams, S. G. Hindes, Loren E. Hunt, B. Bienenfeld, J. P. Carlin, Charles E. Goad, R. B. Green, C. E. Moore, A. L. A. Himmelwright, Langdon Pearse, Charles Derleth, Jr., and J. D. Galloway, 264.

ELECTRIC LIGHTING.

Effect of San Francisco earthquake on — plants. 257.

ELECTRIC RAILWAYS.

"Electric Railways." An Informal Discussion. George Gibbs, W. W. Follett, and J. T. Noble Anderson. 330.

ENGINES.

"Gas —." An Informal Discussion. James Christie, Robert Heywood Fernald, Timoleon Geelen, and J. M. Buchanan. 402.

FILTRATION.

"Water Supply." An Informal Discussion. George W. Fuller and others. 367.

FIRE PROOFING.

"The Effects of the San Francisco Earthquake of April 18th, 1906, on Engineering Constructions": Reports of a General Committee and of Six Special Committees of the San Francisco Association of Members of the American Society of Civil Engineers. (With Discussion.) 208.

FIRE PROTECTION.

"The Effects of the San Francisco Earthquake of April 18th, 1906, on Engineering Constructions": Reports of a General Committee and of Six Special Committees of the San Francisco Association of Members of the American Society of Civil Engineers. (With Discussion.) 208.

GAS.

Effect of San Francisco earthquake on — plants. 256.

"— Engines." An Informal Discussion. James Christie and others. 402.

GEOLOGY.

"The Effects of the San Francisco Earthquake of April 18th, 1906, on Engineering Constructions": Reports of a General Committee and of Six Special Committees of the San Francisco Association of Members of the American Society of Civil Engineers. (With Discussion.) 208.

"The Lower Colorado River and the Salton Basin." C. E. Grunsky. (With Discussion.) 1.

IRRIGATION.

"The Lower Colorado River and the Salton Basin." C. E. Grunsky. (With Discussion.) 1.

LEVEES.

"The Lower Colorado River and the Salton Basin." C. E. Grunsky. (With Discussion.) 1.

MEMOIRS OF DECEASED MEMBERS.

Abert, Sylvanus Thayer. 521.
Blackwell, Charles. 531.
Bouscaren, Louis Gustave Frederic. 533.
Campbell, Albert Johnstone. 563.
Cheney, John Eugene. 537.
Guild, Josephus Conn. 540.
Hemming, Dunkin Wirgman. 543.
Hequembourg, Charles Ezra. 545.
Linville, Jacob Hays. 549.
Nicholson, George Benson. 556.
Rogers, Merritt Harrison. 562.
Zipperlein, Joseph William. 565.

PAVEMENT.

"Pavements." An Informal Discussion. George W. Tillson, J. H. Haylow, Clifford Richardson, Horace Andrews, Gardner S. Williams, Lansing H. Beach, E. M. T. Ryder, William H. Lawton, Philip W. Henry, B. Bienenfeld, Nelson P. Lewis, and Edward P. North. 336.

PIPE.

"A Solution of the Problem of Determining the Economic Size of — for High-Pressure Water-Power Installation." Arthur L. Adams. (With Discussion.) 173.

PIPE LINES.

Effect of San Francisco earthquake on — . 251.

RAINFALL.

"Comparison between — and Run-Off in the Northeastern United States." John C. Hoyt. 431. Discussion: E. C. Murphy, Charles A. Holden, H. C. Frankenfield and Alfred J. Henry, Thaddeus Merriman, Raphael Zon, C. C. Vermeule, H. K. Barrows, J. C. Stevens, M. O. Leighton, F. H. Newell, A. P. Davis, N. C. Grover, and F. W. Hanna, 484.

REINFORCED CONCRETE.

"The — Bridge across the Hudson River at Sandy Hill, New York." William H. Burr. (With Discussion.) 195.

REPORTS OF COMMITTEES.

"The Effects of the San Francisco Earthquake of April 18th, 1906, on Engineering Constructions": Reports of a General Committee and of Six Special Committees of the San Francisco Association of Members of the American Society of Civil Engineers. (With Discussion.) 208.

RIVERS.

"Comparison between Rainfall and Run-Off in the Northeastern United States." John C. Hoyt. (With Discussion.) 431.

"The Lower Colorado River and the Salton Basin." C. E. Grunsky. 1. Discussion: W. G. Price, Rutger B. Green, T. G. Dabney, W. de H. Washington, and J. A. Ockerson, 52.

RUN-OFF.

"Comparison between Rainfall and — in the Northeastern United States." John C. Hoyt. (With Discussion.) 431.

SEWERAGE.

Effect of San Francisco earthquake on sewers. 214.

SIPHONS.

"Liberation of Air in —." Charles Anthony, Jr. 63. Discussion: P. M. Pritchard, 65.

STREET RAILWAYS.

Effect of San Francisco earthquake on —. 256.

WATER, FLOW OF, IN PIPES.

"A Solution of the Problem of Determining the Economic Size of Pipe for High-Pressure Water-Power Installation." Arthur L. Adams. 173. Discussion: W. L. Butcher, W. E. Buck, P. E. Stevens, C. L. DeMott, E. Kuichling, William J. Roberts, Charles D. Marx and C. B. Wing, James H. Wise, E. W. Myers, Asahel C. Toll, and F. G. Baum, 178.

WATER POWER.

"A Solution of the Problem of Determining the Economic Size of Pipe for High-Pressure — Installation." Arthur L. Adams. (With Discussion.) 173.

WATER-PROOFING.

Permeability of concrete and methods of —. 159.

Permeability tests of concrete. 127.

WATER-WORKS.

"The Effects of the San Francisco Earthquake of April 18th, 1906, on Engineering Constructions": Reports of a General Committee and of Six Special Committees of the San Francisco Association of Members of the American Society of Civil Engineers. (With Discussion.) 208.

"Water Supply." An Informal Discussion. George W. Fuller, F. Herbert Snow, D. D. Clarke, George A. Soper, William J. Baldwin, Gardner S. Williams, J. T. Noble Anderson, William B. Fuller, J. Waldo Smith, Pablo Solís and Octavio Guzmán, S. Bent Russell, M. R. Sherrerd, F. W. Blackford, George A. Johnson, and George W. Fuller. 367.

AUTHOR INDEX.

ABERT, SYLVANUS THAYER.
Memoir of. 521.

ADAMS, ARTHUR L.
"A Solution of the Problem of Determining the Economic Size of Pipe for High-Pressure Water-Power Installation." 173.
Effects of San Francisco earthquake on engineering constructions. 208, 245, 285.

ANDERSON, J. T. NOBLE.
Electric railways. 334.
Water supplies. 389.

ANDREWS, HORACE.
Pavements. 346.

ANTHONY, CHARLES, Jr.
"Liberation of Air in Siphons." 63.

BALDWIN, WILLIAM J.
Water supplies. 388.

BARROWS, H. K.
Rainfall and stream flow. 499.

BAUM, F. G.
Size of high-pressure water-power pipe. 193.

BEACH, LANSING H.
Pavements. 349.

BIENENFELD, A. M.
Effects of San Francisco earthquake on engineering constructions. 284.

BIENENFELD, BERNARD.
Effects of San Francisco earthquake on engineering constructions. 285.
Pavements. 361.

BLACKFORD, F. W.
Water supplies. 398.

BLACKWELL, CHARLES.
Memoir of. 531.

BOGGS, E. M.
Effects of San Francisco earthquake on engineering constructions. 258.

BOUSCAREN, LOUIS GUSTAVE FREDERIC.
Memoir of. 533.

BUCHANAN, J. M.
Gas engines. 428.

BUCK, W. E.
Size of high-pressure water-power pipe. 179.

BURR, WILLIAM H.
"The Reinforced Concrete Bridge across the Hudson River at Sandy Hill, New York." 195.

BUTCHER, W. L.
Size of high-pressure water-power pipe. 178.

CAMPBELL, ALBERT JOHNSTONE.
Memoir of. 563.

CARLIN, J. P.
Effects of San Francisco earthquake on engineering constructions. 286.

CHENEY, JOHN EUGENE.
Memoir of. 537.

CHRISTIE, JAMES.
Gas engines. 402.

CLARKE, D. D.
Water supplies. 384.

COUCHOT, M. C.
Effects of San Francisco earthquake on engineering constructions. 223.

DABNEY, T. G.
Colorado River and Salton Basin. 54.

DAVIS, A. P.
Rainfall and stream flow. 509.

DAVIS, JAMES L.
Laws of proportioning concrete. 144.

DeMOTT, C. L.
Size of high-pressure water-power pipe. 183.

DENNIS, W. F.
Laws of proportioning concrete. 157.

DERLETH, CHARLES, Jr.
Effects of San Francisco earthquake on engineering constructions. 223, 311.

DILLMAN, GEORGE L.
Effects of San Francisco earthquake on engineering constructions. 245.

DRAKE, R. M.
Effects of San Francisco earthquake on engineering constructions. 258.

DURYEA, EDWIN, Jr.
Effects of San Francisco earthquake on engineering constructions. 208, 264.

FERET, R.
Laws of proportioning concrete. 152.

FERNALD, ROBERT HEYWOOD.
Gas engines. 405.

FOLLETT, W. W.
Electric railways. 333.

FRANKENFIELD, H. C.
Rainfall and stream flow. 489.

FREITAG, J. K.
Effects of San Francisco earthquake on engineering constructions. 277.

FULLER, GEORGE W.
Water supplies. 367, 400.

FULLER, WILLIAM B.
"The Laws of Proportioning Concrete." 67.
Water supplies. 390.

GAINES, RICHARD H.
Laws of proportioning concrete. 159.

GALLOWAY, J. D.
Effects of San Francisco earthquake on engineering constructions. 223, 323.

GEELEN, TIMOLEON.
Gas engines. 413.

GIBBS, GEORGE.
Electric railways. 330.

GILMAN, C. E.
Effects of San Francisco earthquake on engineering constructions. 216, 245.

GOAD, CHARLES E.
Effects of San Francisco earthquake on engineering constructions. 289.

GREEN, RUTGER B.
Colorado River and Salton Basin. 53.
Effects of San Francisco earthquake on engineering constructions. 292.

GROVER, N. C.
Rainfall and stream flow. 510.

GRUNSKY, C. E.
"The Lower Colorado River and the Salton Basin." 1.

GUILD, JOSEPHUS CONN.
Memoir of. 540.

GUZMÁN, OCTAVIO.
Water supplies. 392.

HAAS, E. F.
Effects of San Francisco earthquake on engineering constructions. 263.

HAEHL, H. L.
Effects of San Francisco earthquake on engineering constructions. 216.

HANNA, F. W.
Rainfall and stream flow. 511.

HARROUN, PHILIP E.
Effects of San Francisco earthquake on engineering constructions. 245.

HARTS, W. W.
Effects of San Francisco earthquake on engineering constructions. 208, 282.

HAYLOW, J. H.
Pavements. 341.

HAZEN, ALLEN.
Laws of proportioning concrete. 149.

HEMMING, DUNKIN WIRGMAN.
Memoir of. 543.

HENRY, ALFRED
Rainfall and stream flow. 489.

HENRY, PHILIP W.
Pavements. 356.

HEQUEMBOURG, CHARLES EZRA.
Memoir of. 545.

HIMMELWRIGHT, A. L. A.
Effects of San Francisco earthquake on engineering constructions. 296.

HINDES, S. G.
Effects of San Francisco earthquake on engineering constructions. 285.

HOLDEN, CHARLES A.
Rainfall and stream flow. 488.

HOYT, JOHN C.
"Comparison between Rainfall and Run-Off in the Northeastern United States." 431.

HUNT, A. M.
Effects of San Francisco earthquake on engineering constructions. 256.

HUNT, LOREN E.
Effects of San Francisco earthquake on engineering constructions. 285.

HYDE, CHARLES GILMAN.
Effects of San Francisco earthquake on engineering constructions. 245.

JOHNSON, GEORGE A.
Water supplies. 398.

KRELLWITZ, D. W.
Reinforced concrete bridges. 206.

KUICHLING, E.
Size of high-pressure water-power pipe. 184.

LAWTON, WILLIAM H.
Pavements. 353.

LEIGHTON, M. O.
Rainfall and stream flow. 504.

LEWIS, NELSON P.
Pavements. 362.

LINVILLE, JACOB HAYS.
Memoir of. 549.

MANSON, MARSDEN.
Effects of San Francisco earthquake on engineering constructions. 263.

MARX, CHARLES D.
Effects of San Francisco earthquake on engineering constructions. 208, 245.
Size of high-pressure water-power pipe. 188.

MERRIMAN, THADDEUS.
Rainfall and stream flow. 491.

MOORE, C. E.
Effects of San Francisco earthquake on engineering constructions. 245, 263, 293.

MURPHY, E. C.
Rainfall and stream flow. 484.

MYERS, E. W.
Size of high-pressure water-power pipe. 191.

NEWELL, F. H.
Rainfall and stream flow. 508.

NICHOLSON, GEORGE BENSON.
Memoir of. 556.

NORTH, EDWARD P.
Pavements. 364.

OCKERSON, J. A.
Colorado River and Salton Basin. 58.

PABLO, SOLÍS.
Water supplies. 392.

PARSONS, H. de B.
Laws of proportioning concrete. 149.

PEARSE, LANGDON.
Effects of San Francisco earthquake on engineering constructions. 310.

PERROT, EMILE G.
Effects of San Francisco earthquake on engineering constructions. 270.

PHILLIPS, H. C.
Effects of San Francisco earthquake on engineering constructions. 258.

PRICE, W. G.
Colorado River and Salton Basin. 52.

PRITCHARD, P. M.
Liberation of air in siphons. 65.

QUIMBY, HENRY H.
Reinforced concrete bridges. 203.

RICHARDSON, CLIFFORD.
Pavements. 342.

RIFFLE, FRANKLIN.
Effects of San Francisco earthquake on engineering constructions. 208, 245, 266.

RIX, E. A.
Effects of San Francisco earthquake on engineering constructions. 283.

ROBERTS, WILLIAM J.
Size of high-pressure water-power pipe. 187.

ROGERS, MERRITT HARRISON.
Memoir of. 562.

RUSSELL, S. BENT.
Water supplies. 394.

RYDER, E. M. T.
Pavements. 351.

SHERRERD, M. R.
Water supplies. 396.

SLOCUM, C. L.
Reinforced concrete bridges. 204.

SMITH, J. WALDO.
Water supplies. 391.

SNOW, F. HERBERT.
Water supplies. 376.

SNYDER, C. H.
Effects of San Francisco earthquake on engineering constructions. 223.

SOPER, GEORGE A.
Water supplies. 387.

STERN, EUGENE W.
Effects of San Francisco earthquake on engineering constructions. 273.
Reinforced concrete bridges. 206.

STEVENS, J. C.
Rainfall and stream flow. 502.

STEVENS, P. E.
Size of high-pressure water-power pipe. 182.

THOMPSON, SANFORD E.
"The Laws of Proportioning Concrete." 67.

TILLSON, GEORGE W.
Pavements. 336.

TOLL, ASAHEL C.
Size of high-pressure water-power pipe. 192.

VERMEULE, C. C.
Rainfall and stream flow. 496.

VON GELDERN, OTTO.
Effects of San Francisco earthquake on engineering constructions. 216.

WADSWORTH, H. H.
Effects of San Francisco earthquake on engineering constructions. 270.

WAGNER, SAMUEL TOBIAS.
Laws of proportioning concrete. 150.

WAGONER, LUTHER.
Effects of San Francisco earthquake on engineering constructions. 216, 272.

WALLACE, J. H.
Effects of San Francisco earthquake on engineering constructions. 258.

WASHINGTON, W. de H.
Colorado River and Salton Basin. 55.

WHITAKER, W. F.
Effects of San Francisco earthquake on engineering constructions. 284.

WILLIAMS, GARDNER S.

Pavements. 348.
Water supplies. 388.

WING, CHARLES B.

Effects of San Francisco earthquake on engineering constructions. 223, 245.
Size of high-pressure water-power pipe. 188.

WISE, JAMES H.

Size of high-pressure water-power pipe. 189.

ZIPPERLEIN, JOSEPH WILLIAM.

Memoir of. 565.

ZON, RAPHAEL.

Rainfall and stream flow. 493.